4. Computer Simulation of Coastal Hydrodynamic Processes

Numerical modeling was used to simulate a wide range of coastal processes that affect ferry operations at Keystone Harbor and/or may be affected by them (ferry operations and structures that support operations). The simulated processes included tidal current circulation, wave propagation, refraction, diffraction, and reflection, sediment transport, bottom accretion and erosion, and shoreline changes. Computer simulations of these processes were conducted for existing conditions (No Action alternative) and for each of the proposed alternatives presented in Section 3. Simulations were conducted for a range of input parameters to which the alternatives are most sensitive and most strongly indicate possible changes and effects from the alternatives. Models descriptions, input parameters, and modeling results are discussed below in appropriate sections.

4.1. Tidal Current Circulation Model

The main objective of tidal current circulation modeling was to evaluate performance of proposed project alternatives in their ability to change cross-current velocities and improve navigation conditions at Keystone Harbor. The modeling goals also included evaluation of potential effects from alternatives on coastal processes that may contribute to environmental impacts.

4.1.1. Model Description

Tidal current circulation modeling was performed using the Advanced Circulation Model (ADCIRC) developed by the U.S. Army Corps of Engineers (Luettich *et al* 1992). ADCIRC is a highly developed computer program traditionally used for large scale, region-size projects.

ADCIRC simulates depth-averaged current velocities and directions in the modeling domain forced by tidal fluctuations at the boundaries of the modeling domain. Boundary tidal fluctuations are prescribed using variable tidal constituents introduced along the offshore boundary. Therefore the modeling domain extends far from the project site to avoid boundary effects on the modeling results. Figure 4.1 shows the modeling domain with depths represented by color contours. ADCIRC modeling includes construction of the finite element modeling domain, development of boundary and input parameters, model verification, simulations of modeling scenarios, and analysis of the output parameters.

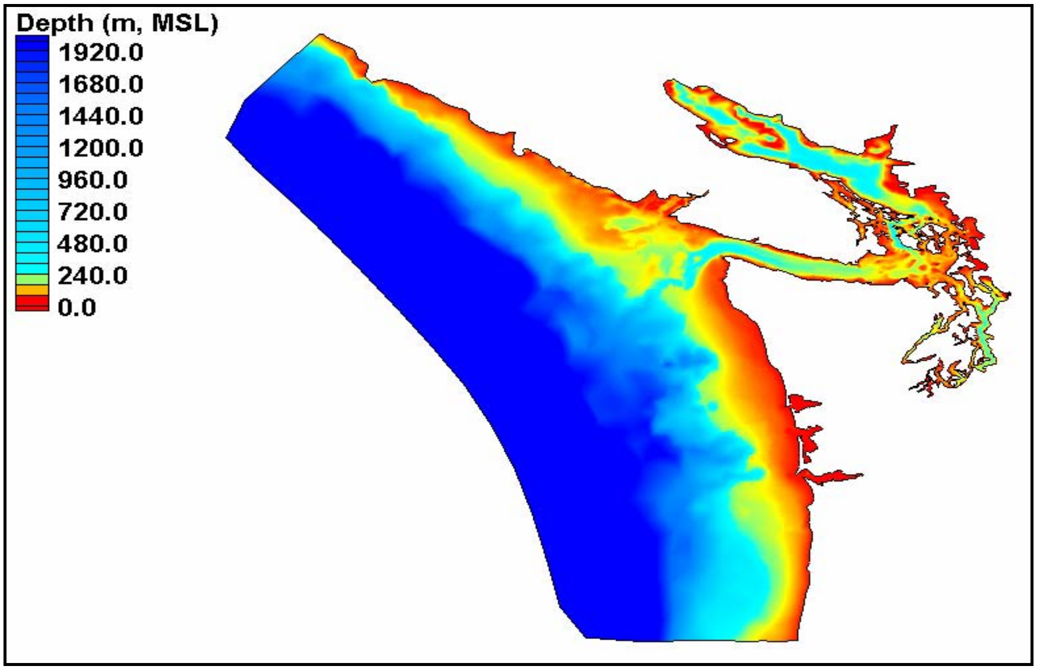

Figure 4.1 Existing Conditions Modeling Domain

4.1.2. Modeling Domain and Grid

Keystone Harbor numerical modeling domain covers the area offshore from the Straight of Juan de Fuca at approximately 360 miles and most of Puget Sound. Construction of modeling grid required collection, processing, compilation and analysis of detailed bathymetry data for the entire domain. Various sources of data were used to compile the modeling domain grid. The sources included bathymetric surveys from Washington State Ferries, compiled bathymetry and topography from the University of Washington, and shoreline data from the coast of Washington, Vancouver Island and all of Puget Sound digitized from rectified digital orthogonal quadrangle photos (United States Geological Survey 1990) and others. Numerous iterations were attempted using various grid geometrical configurations until a domain was developed that was proven to simulate tide wave transformation processes in an effective manner. The size of the modeling finite element mesh varies along the modeling domain and changes from several miles at offshore part of the grid to several meters in the vicinity of the project area. The model domain consists of 68,672 elements and 37,414 nodes. The modeling finite element mesh is shown in Figure 4.2 for the entire domain and in Figure 4.3 in a close-up of the project area with color depth contours.

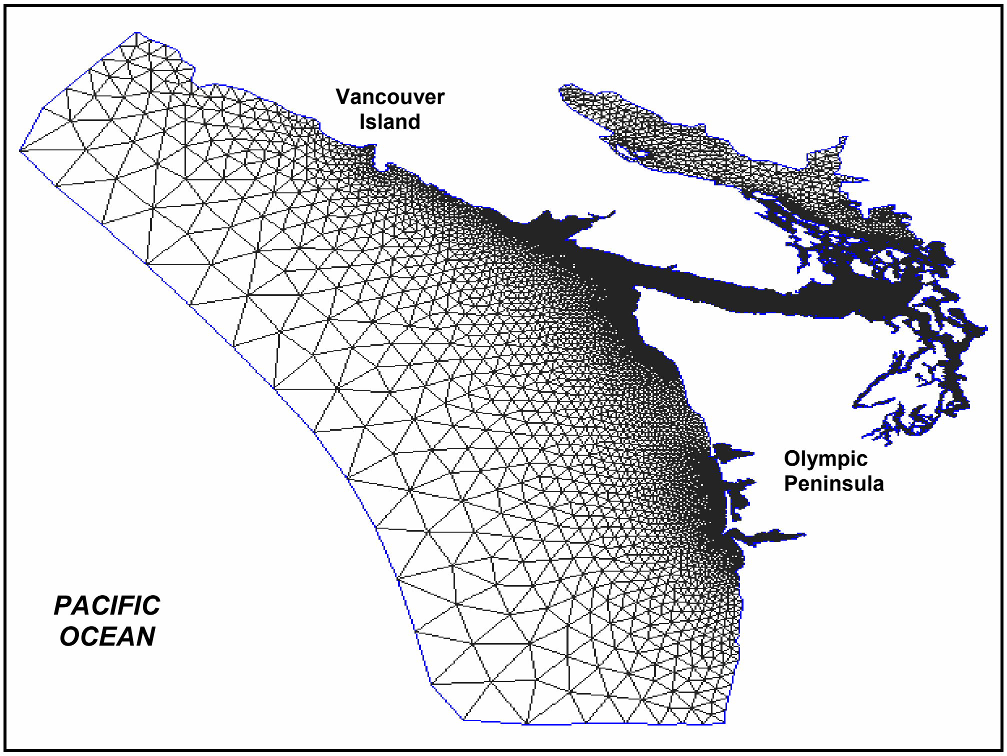

Figure 4.2 Finite Element Modeling Grid for Entire Model Domain

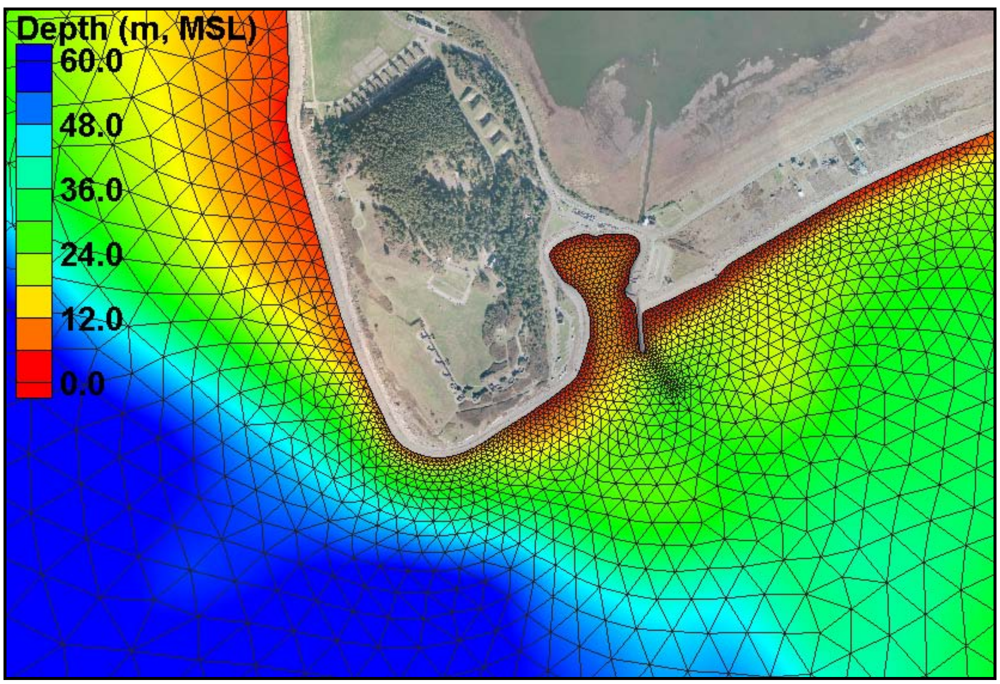

Figure 4.3 Modeling Domain Finite Element Grid Close-Up at Project Site

4.1.3. Boundary and Input Conditions

Modeling of large-scale regional processes requires boundary condition model forcing information over a large area of the ocean. In this case, tidal elevation boundary conditions were required over the Pacific Ocean and Puget Sound areas at a distance greater than 590 km (367 miles). Tidal constituents developed during global water level modeling studies (LeProvost *et al* 1994) were used to generate time series of water level at each ocean boundary node for the purpose of driving the model. Model input conditions consist of several computational parameters and physical constants. One of the variable and controlling physical constants is a bottom friction parameter. Bottom friction that was used during Keystone study was the default value of 0.0025 for all areas.

The model simulated currents and tidal fluctuations over a ten-day period during of the 2004 field data collection program (02/28/04 00:00 UTC to 03/09/04 00:00 UTC). The modeling period included a range of tidal fluctuations that typically occur at the project area. Figure 4.4 shows the statistical distribution of tidal ranges at Admiralty Head (NOAA Station #993, 122.6667° W 48.1667° N) over the last 10-year period. Figure 4.4 also include values of tidal ranges predicted at Admiralty Head during the simulation period. It can be seen that during the simulation period, large tidal ranges and small tidal ranges occurred more frequently than the long-term average in the tidal record.

Medium-size tidal ranges occurred less frequently than in the long-term record. This is clear from the shape of the tides measured during the simulation, as there were very small ranges present with larger ranges. Based on the statistical data and analysis it is suggested that the modeling period reasonably represents typical tidal fluctuations at the project site. The ADCIRC model was setup to calculate depth-averaged current velocities and water surface elevations at each node within the domain at a frequency of 2 Hz (twice per second).

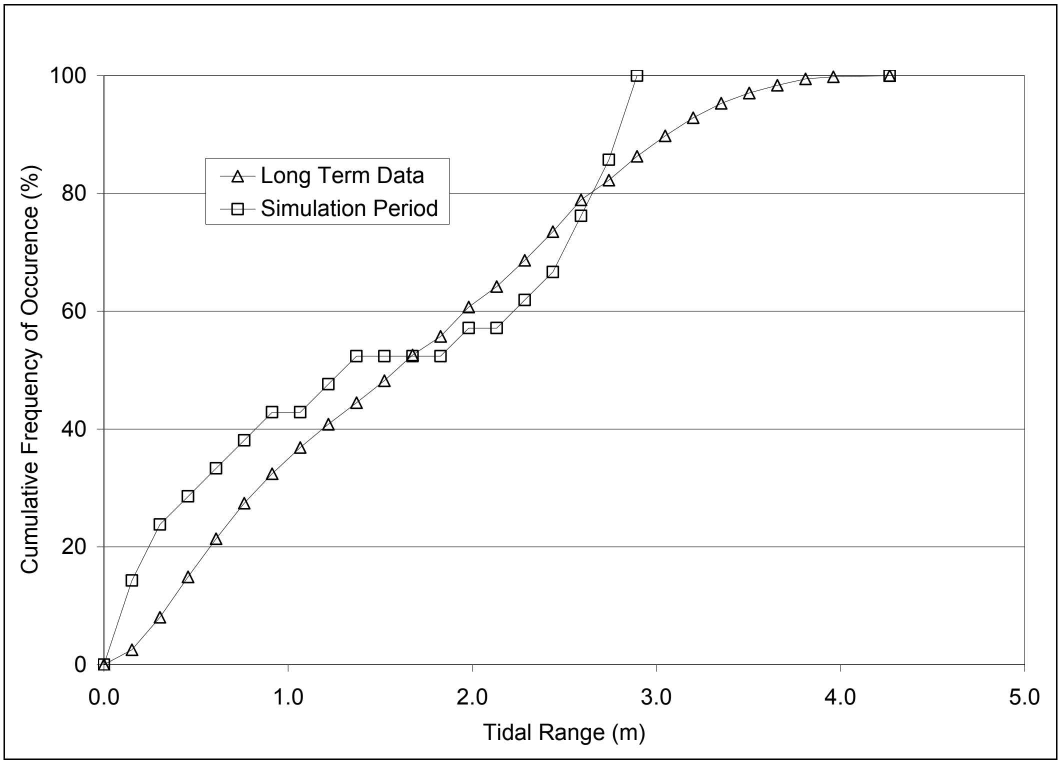

Figure 4.4 Statistical Distribution of Tidal Ranges during Simulation Period and Long-Term at Admiralty Head

4.1.4. Model Verification

Model verification was conducted for existing conditions using general knowledge of tidal currents from previous studies and direct field measurements of tidal elevations and current velocities during 2004 field data collection program. In addition, data obtained during physical modeling (see Section 5) were applied to validate both the physical and numerical (ADCIRC) models in an attempt to achieve consistency in the modeling results.

4.1.4.1 Verification Based on Information from Previous Studies

Previous studies have shown that the tidal current flow patterns in Admiralty Bay and Admiralty Inlet are extremely complex and variable. During both ebb and flood tide, eddies form downstream of flow-obstructing headlands at Admiralty Head and across Admiralty Inlet at Point Wilson. The ADCIRC numerical modeling conducted for the existing conditions demonstrates the complexity of the current patterns discussed above. Figure 4.5 shows the results of the modeling for typical ebb flow conditions at four snapshots (four instants during the simulation) during the ebb cycle for existing conditions. Figure 4.6 shows the results of ADCIRC modeling for typical flood flow conditions at four snapshots during the flood cycle for existing conditions. During flood tide, eddies form downstream of the surrounding headlands and strip off and travel downstream with the flow, resulting in a constantly varying flow field within Admiralty Inlet and Admiralty Bay during the entire tidal cycle.

These patterns are consistent with past field measurements and observations in Admiralty Inlet, which indicated that cross-channel variability at Admiralty Inlet required multiple mooring sites across the channel to resolve the variations of the along-channel flow through the sections (Cannon *et al* 2001). Past studies have also noted the high variability in the circulation in the eastern Strait of Juan de Fuca, including complex patterns of eddies, fronts, and shore-directed current components, which tend to increase with eastward distance into the system towards Admiralty Inlet (Holbrook *et al* 1980). Figure 4.7 shows typical flood current patterns in Admiralty Inlet taken from a physical model of Puget Sound (Cannon *et al* 1978). These patterns in general compare well with the flood current patterns predicted by the ADCIRC model shown in Figure 4.6.

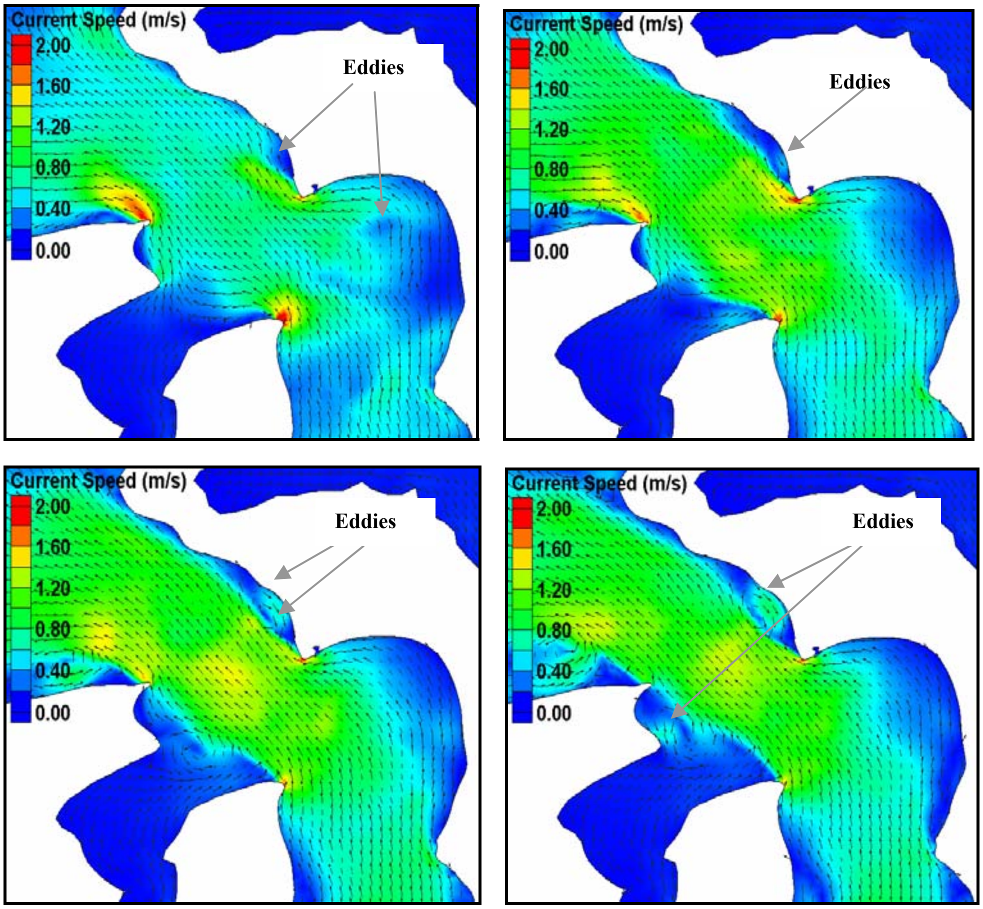

Figure 4.5 Ebb Currents at Four Stages during the Ebb Tidal Cycle (top left to bottom right: 03/03/04 23:00, 03/04/04 00:00, 03/04/04 01:00, 03/04/04 02:00 UTC)

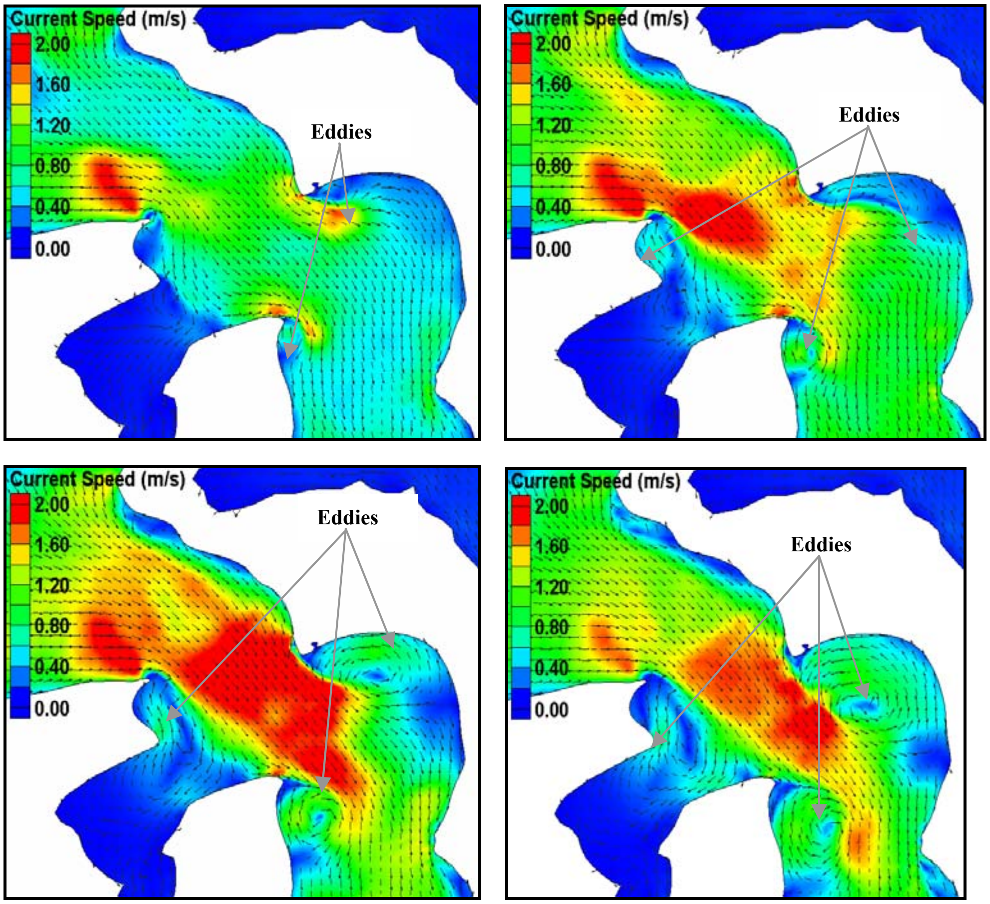

Figure 4.6 Flood Currents at Four Stages during the Flood Tidal Cycle (top left to bottom right: 03/04/04 06:30, 03/04/04 07:30, 03/04/04 08:30, 03/04/04 09:30 UTC)

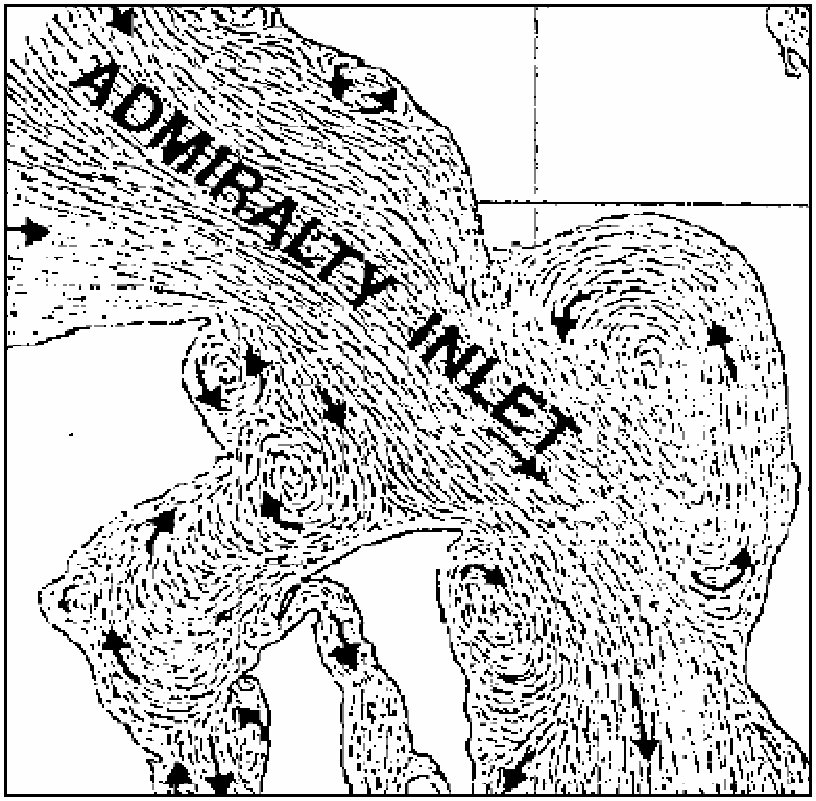

Figure 4.7 Flood Currents in Physical Model of Puget Sound at University of Washington, Seattle, WA (from Cannon et al 1978)

4.1.4.2 Verification Based on Field Measurements

Field measurements of tidal elevations and current velocities collected during the field data collection program in February and March 2004 were used to verify the ADCIRC model. Model verification was conducted by comparison of field data with the modeling results. Calibration (if required) includes adjustment of model parameters until correlation between field data and modeling results becomes sufficient.

Field data from three gauges (location of the gauges is shown in Figure 2.3, see Section 2.2) were compared with the modeling results[4]. It should be noted that the model was started with zero water level and zero velocities in the domain (cold start), requiring significant spin-up time until calculated conditions became reliable. Therefore, even though the model simulation period was ten days, the verification utilizes only the final seven days of the simulation. The ADCIRC model validation was conducted separately for computed tidal elevations and current velocities.

[4] Gauges were installed in the model at the same location where the gauges were installed in the field.

The comparison between measured and computed tidal elevations at Station A is shown in Figure 4.8, which demonstrates a reasonable correlation between these two datasets. The same reasonable level of correlation between modeled and measured data is observed at Stations B and C. Although some differences occur in calculated and measured tidal elevations, based on the comparison it was concluded that the model should be considered validated with regard to tidal elevation.

Figures 4.9, 4.10, and 4.11 show measured and calculated current speeds at Stations A, B, and C, respectively. The results of the comparison show a reasonable correlation between measured and calculated current velocities at Stations A and B and weak correlation at Station C. Station C is located in an area of highly variable eddying and current fluctuation in the model. In addition, Station C is located at the area without recent hydrographic survey data. Model geometrical accuracy is the most important factor controlling accuracy of model predictions. It is possible that changes in depth have occurred during the period since the survey that may affect the modeling results. Considering the reasonable correlation between measured and calculated currents at Stations A and B, as well as the distance of Station C from the project area, it is suggested that the ADCIRC model is verified with respect to current velocities near the project area.

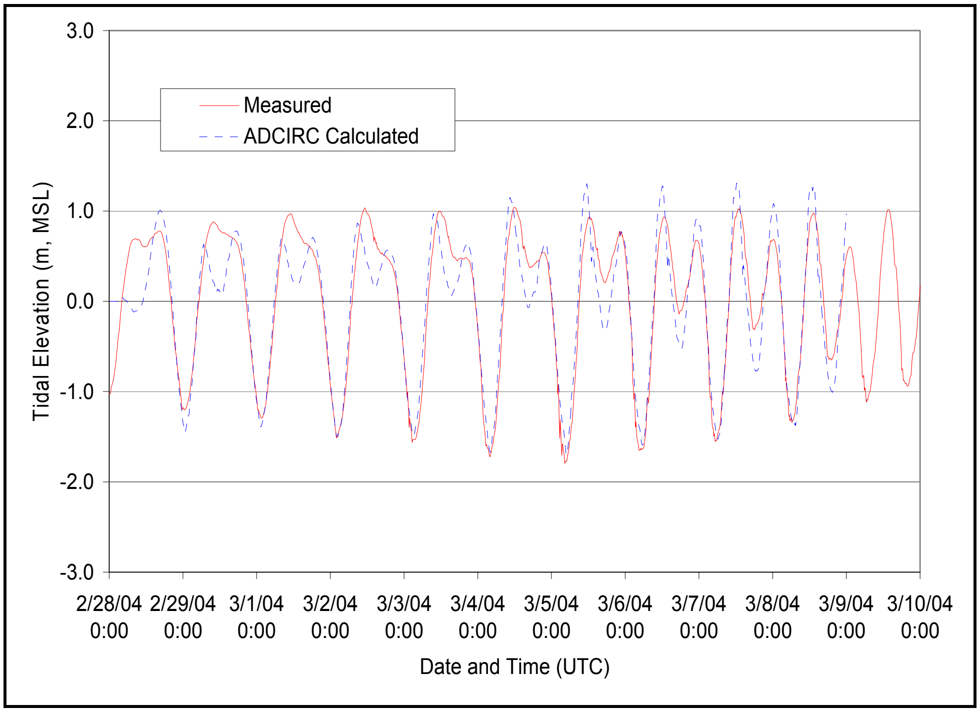

Figure 4.8 Comparison of Measured and Calculated Tidal Elevations at Station A

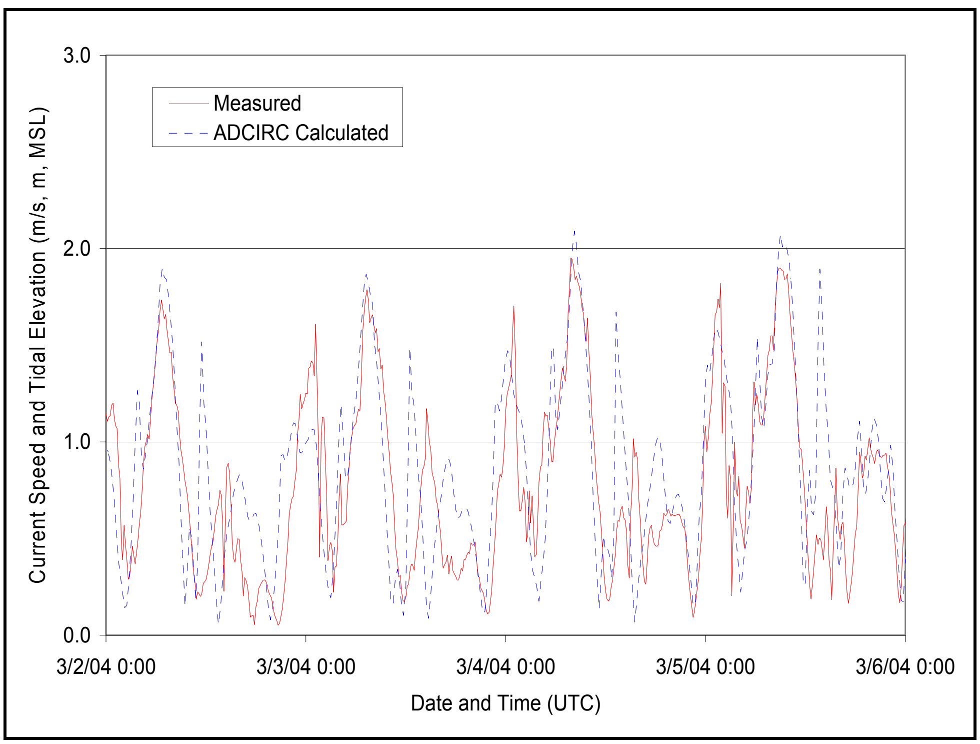

Figure 4.9 Comparison of Measured and Calculated Current Speeds at Station A

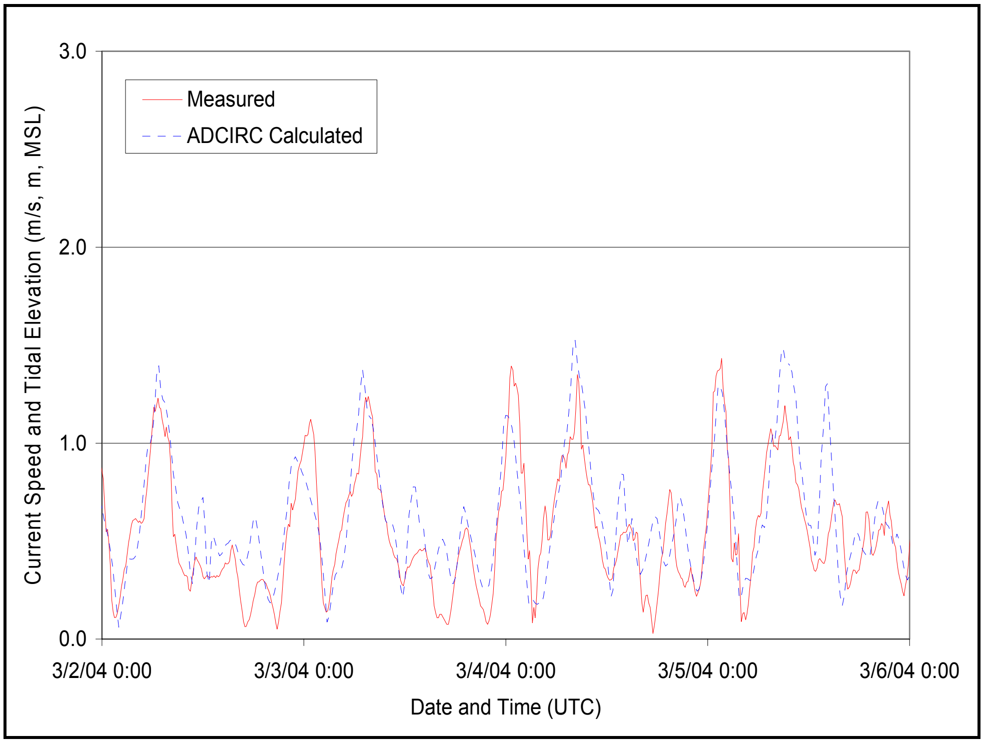

Figure 4.10 Comparison of Measured and Calculated Current Speeds at Station B

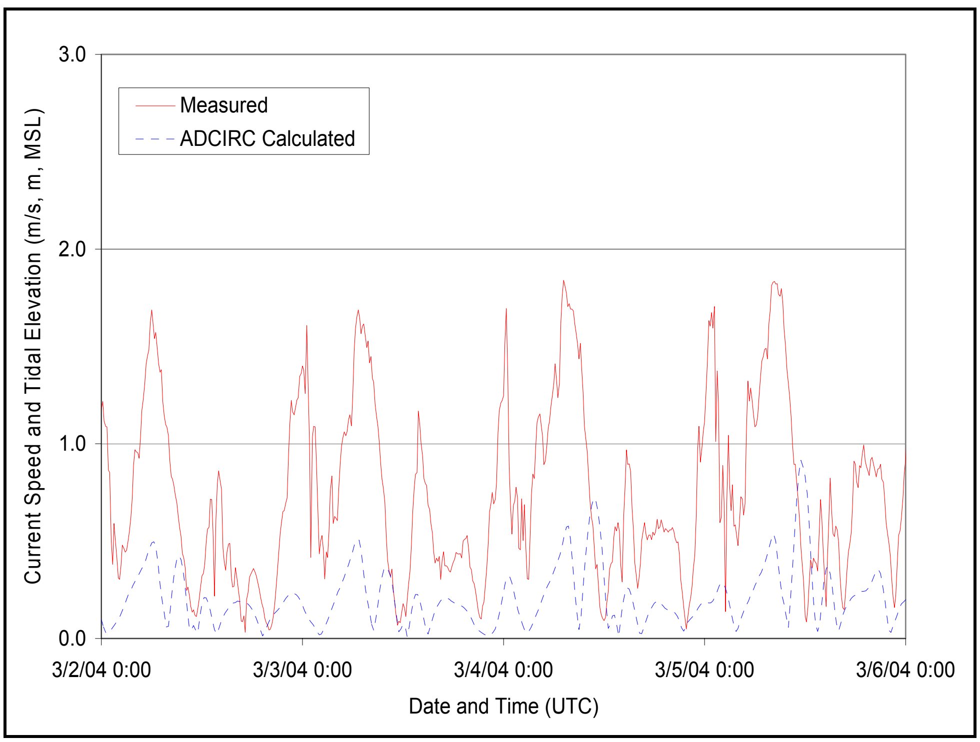

Figure 4.11 Comparison of Measured and Calculated Current Speeds at Station C

4.1.4.3 Verification Based on Physical Modeling

Computer modeling was conducted to simulate the current velocities measured in the physical model. Numerical grid of the physical model was reconstructed in the computer using physical model LIDAR survey. It should be noted that the physical model simulates steady ebb current conditions. Therefore, a steady flow current model was constructed with the same geometry as included in the physical model.

Physical modeling results for existing conditions, extracted from Section 5 are used herein for comparison and validation the computer modeling results. Figure 4.12 shows in color the current velocities at both physical model and computer model. The comparison of these figures was conducted and the results of comparison are shown in Figure 4.13.

Overall, the numerical model results matched those obtained through physical modeling in the vicinity of the channel near the jetty. The differences between numerical modeling and physical modeling are observed at the boundaries of modeling domains and for alternatives consisting of submerged jetty and wave barrier (Alternative 3B and Alternative 4). ADCIRC modeling domain boundaries (See section 4.1.2) are extended far away from the project area. Therefore the differences observed at the physical model boundary do not effect the accuracy of numerical model results. Submerged jetty and wave barrier alternatives results in three-dimensional flow. In the 3D flow cases, the velocities of the ADCIRC model typically under-predicted the velocities to the south of the jetty and over-predicted the velocities in the lee of the jetty. This would result in a conservative estimate of the Alternative's ability to reduce velocities in the navigation channel. The more detailed discussion of comparison physical modeling and computer modeling results is presented in Section 5.9.1.1.

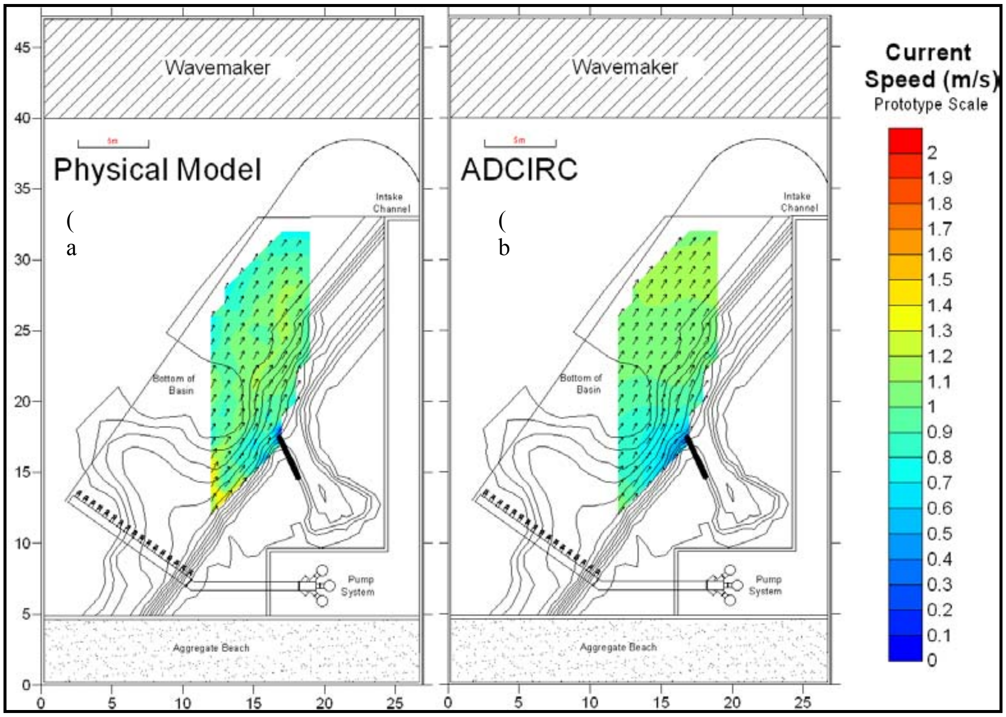

Figure 4.12 Current velocity and direction for existing conditions for (a) physical model and (b) ADCIRC simulation of the physical modeling domain.

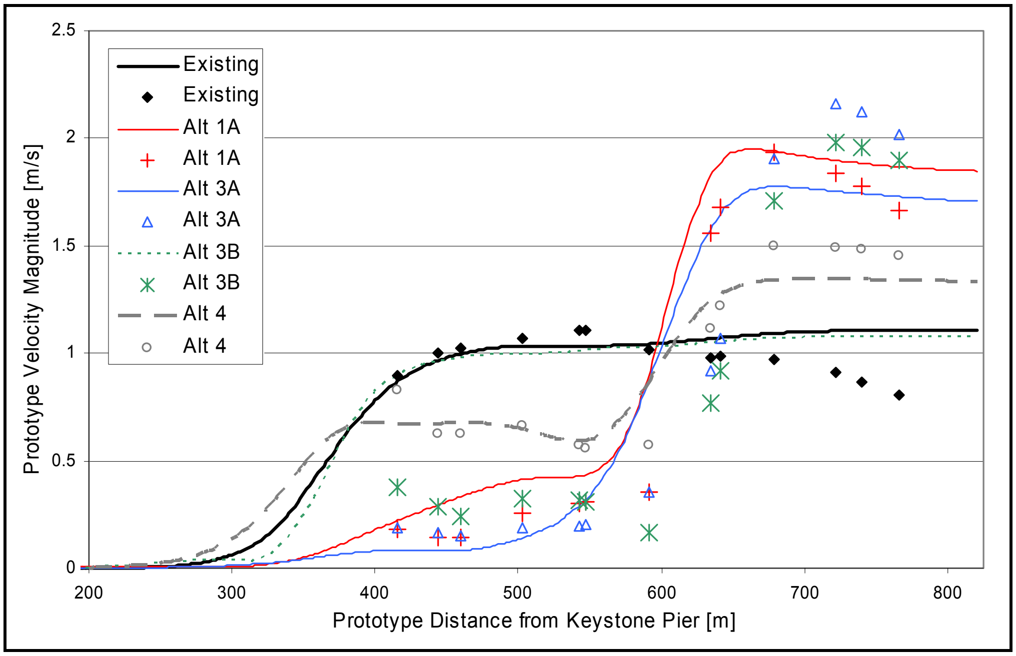

Figure 4.13 Comparison of the physical modeled velocities (points) and the ADCIRC simulated velocities (lines) along the Keystone Channel as a function of distance from the Keystone Pier. All values are in prototype scale.

4.1.5. Modeling Results

The ten project alternatives were constructed into finite element domains similar to the existing conditions domain[5]. Outside the project area, the finite element grids were identical, and incorporated identical boundary conditions and modeling input parameters. Figure 4.14 shows a close-up of the modeling domain and bathymetry for the existing conditions grid. It should be noted that in the following sections, the main report includes only examples of modeling results figures for the No Action Alternative (existing conditions), Alternatives 1, 3, and 5. The full set of figures and data showing the modeling results are provided in Appendix B.

Simulations were performed for each project alternative using the same input parameters as those used to verify the model. Ten-day simulations were performed, and the results were analyzed over the final seven days to eliminate spin-up error. In order to quantify the difference in the modeling results, five measuring stations in the model were established. Figure 4.15 shows instantaneous current speeds and directions for existing conditions during ebb (03/05/04 00:00 UTC) phase. Model results show cross-currents at the South end of the existing jetty fluctuate during the simulation. For typically conditions (shown in the figures above), the current velocities are in the range of 1-1.5 m/s (2-3 knots) during ebb and 0.5-1.0 m/s (1 to 2 knots) during flood. For maximum and extreme conditions, the current velocities may exceed 3.0 meters (6.0 knots) during ebb and 1.5 m/s (3 knots) during flood.

[5] Alternatives 2 and 4 were constructed into the grid using the roughness value from previous study (Luettich *et al* 1999, and Yarnell *et al* 1934a, 1934b). to approximate the flow obstruction effect of the wave barriers.

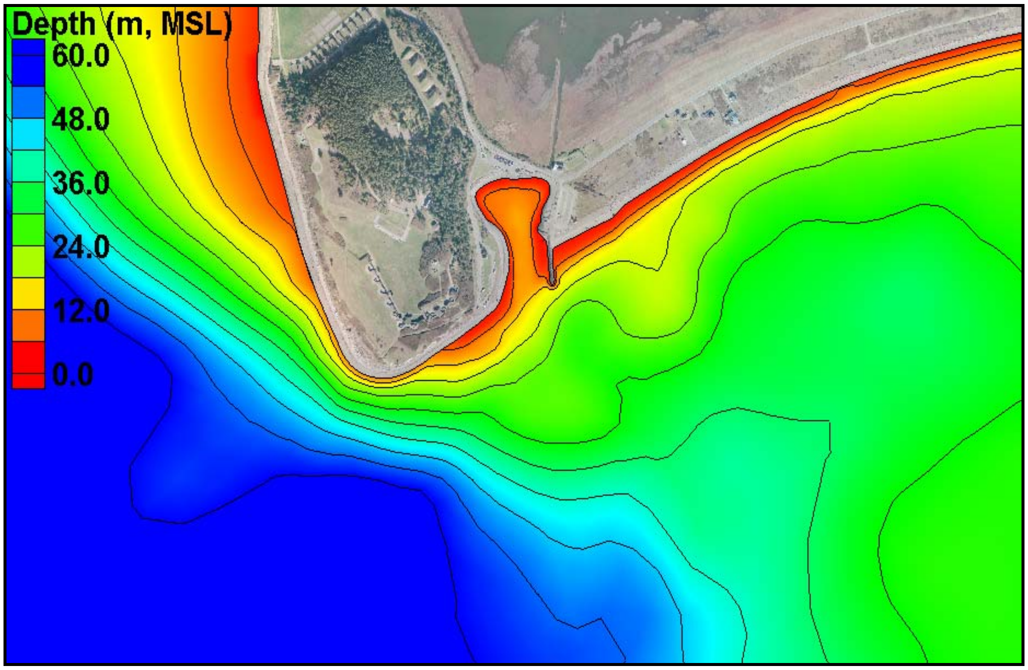

Figure 4.14 Existing Conditions Modeling Domain Close-Up at Project Site

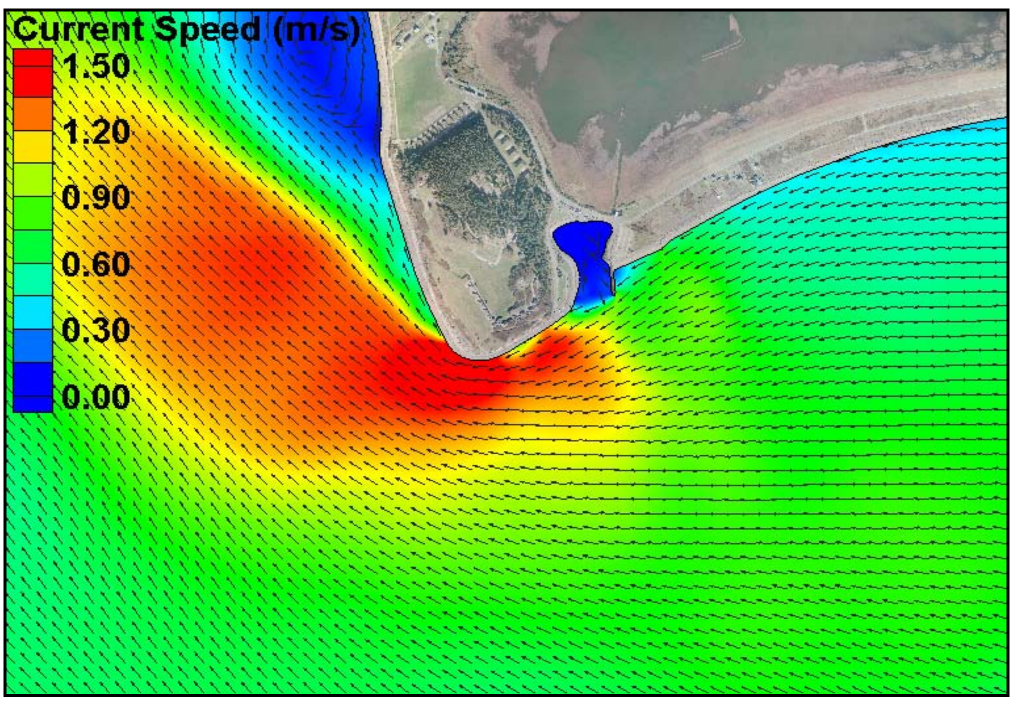

Figure 4.15 Existing Conditions, Ebb Currents (03/05/04 00:00 UTC)

Alternatives evaluation was conducted using two methods: Method 1- comparison of cross–channel current velocities along the channel centerline and Method 2 - comparison of current velocity maps (plan-views) over the modeling domain.

Method 1: Computed current velocities for existing and post-project conditions were extracted along the centerline of the navigation channel (transect). The location of the transect used in analysis is shown in Figure 4.16. Figures 4.17, 4.18, and 4.19 show the cross current velocities along the transect for existing conditions, Alternatives 1, 3 and 5.

Method 2: The spatial distribution of current velocities for the No Action (existing conditions) and other alternatives were plotted and compared. The differences in current velocities between existing conditions and appropriate alternative (No Action "minus" Alternative) were calculated at each element of the modeling grid and 2-Dimensional graphs of differences were plotted. Figures 4.20, 4.21, and 4.22 show the differences in current velocities between Existing conditions and Alternatives 1, 3, and 5 during ebb phase (03/04/04 00:00 UTC). Figure 4.23 shows the differences in current velocities between existing conditions and Alternatives 1 during flood phase (03/04/04 08:15 UTC).

The analysis of modeling results shows that jetty modifications alternatives would provide changes to cross current velocities along the approach to the harbor as follows:

- All jetty extension alternatives (excluding Alternative 3B) would reduce significantly cross current velocities in the channel along the length of the extended jetty at approximately 600 ft along the channel. Reduction of cross current velocities relatively along this stretch of the channel is calculated at approximately 70-80% relatively to existing conditions.
- Alternative 3B would provide smaller reduction of cross current velocities in the channel along the extended jetty. This reduction would not exceed 30% relatively to existing conditions.
- All jetty extension alternatives would provide some reduction of current velocities seaward of the extended jetty at approximately to 20-30% relatively to existing conditions. However, prior to reduction, an increase of velocities may occur immediately at the end of the extended jetty. This immediate increase would increase the shear, specifically for Alternatives 1, 1A, 2 and 2A.
- Jetty Relocation Alternative would provide >30-50% reduction of cross current velocities at distance of more than 2,000 ft of the seaward end of the existing jetty.
- Jetty Relocation Alternative would reduce the shear effect at the entrance of the harbor.

- It is likely that construction of the existing jetty in 1948 has changed the pattern of eddies, exacerbating cross-current velocities at the entrance to the harbor. It is likely that prior to jetty construction, the strength of the currents was smaller and/or had been reduced gradually toward the entrance in accordance to the natural depth. All the jetty modification alternatives would change (to different extent) the tidal flow eddying patterns near the project area. Jetty relocation alternatives would partially re-construct the pre-1948 currents patterns at the entrance to the harbor.

Upon evaluation of the alternatives using two methods, described above and consultations with WSF captains and operational staff it is concluded: From the perspective of cross current velocities controlling factor, two jetty modifications alternatives Alternative 3 (or 3A) –Rock Dogleg and Alternative 5 – Jetty Relocation are feasible. These alternatives are recommended for further analysis.

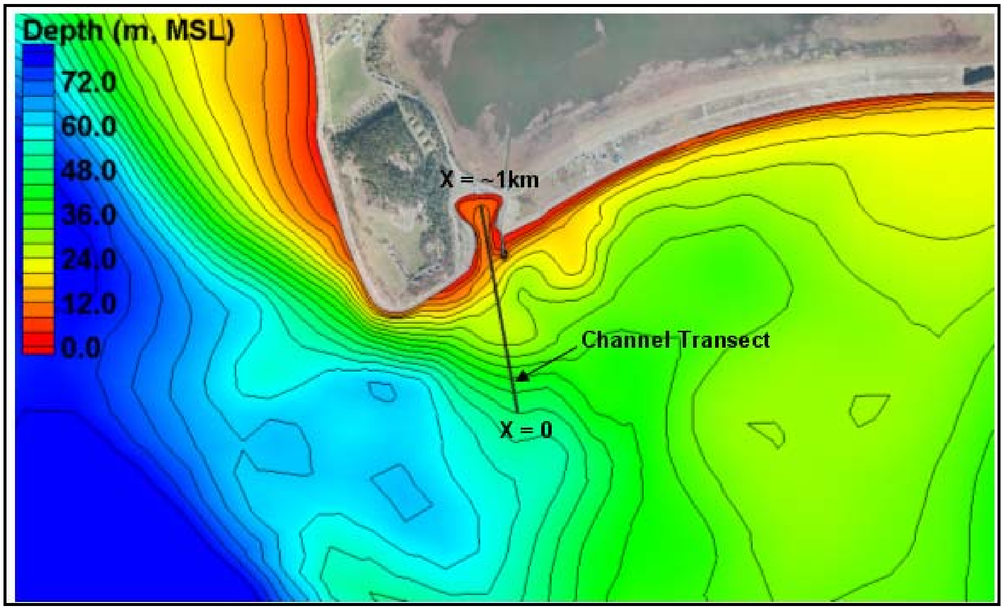

Figure 4-16 Navigation Channel Transect for Current Speed Analysis

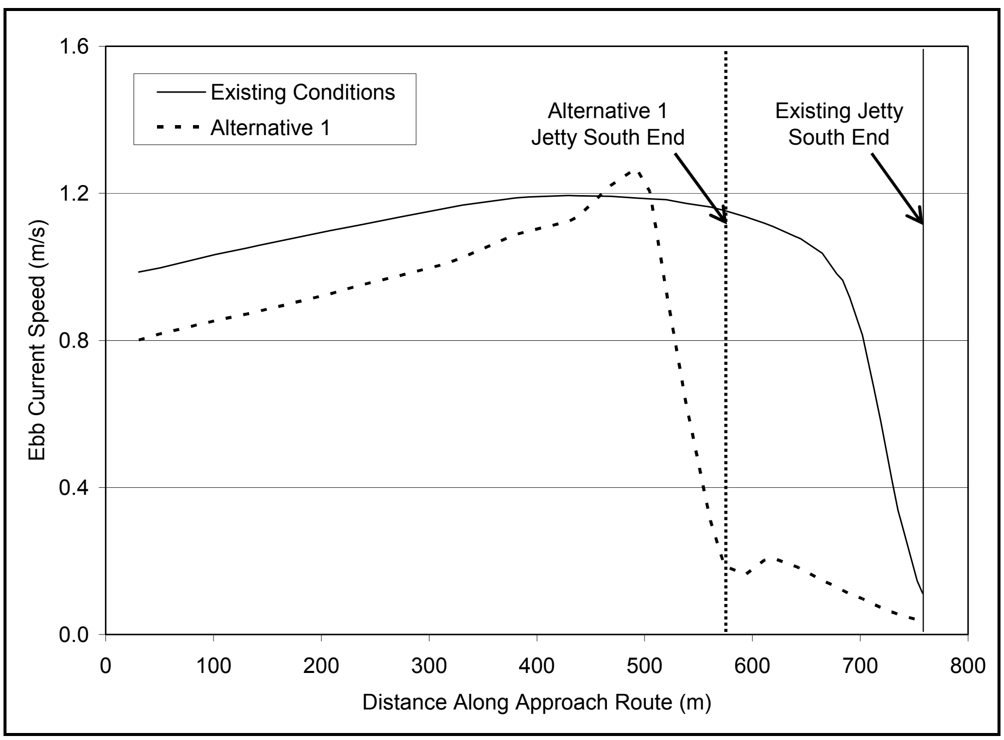

Figure 4-17 Current Speed along Navigation Channel Transect for Existing and Alternative 1 Typical Ebb Conditions

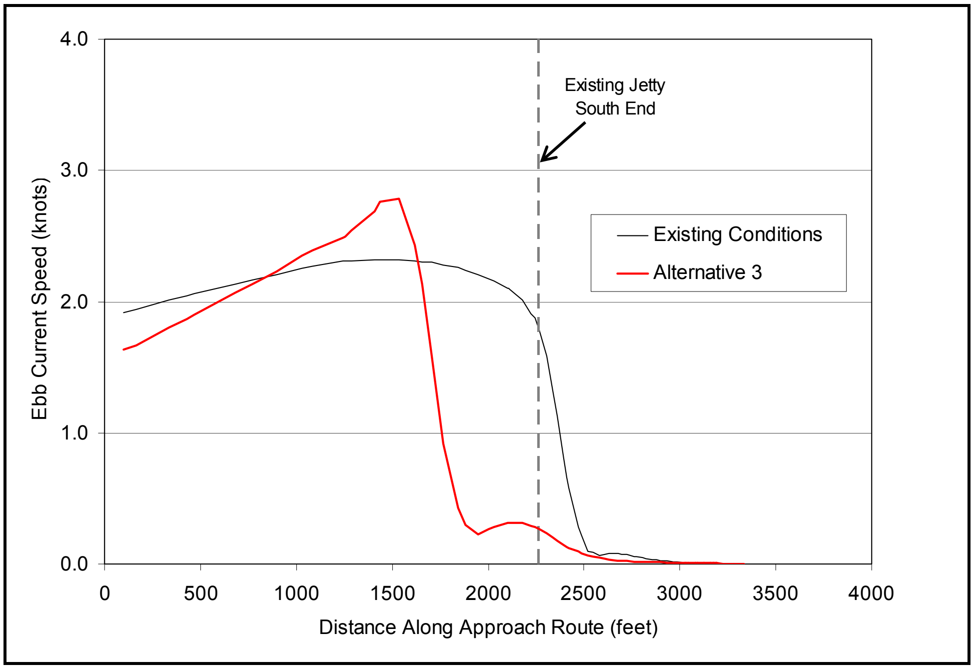

Figure 4-18 Current Speed along Navigation Channel Transect for Existing and Alternative 3 Ebb Conditions

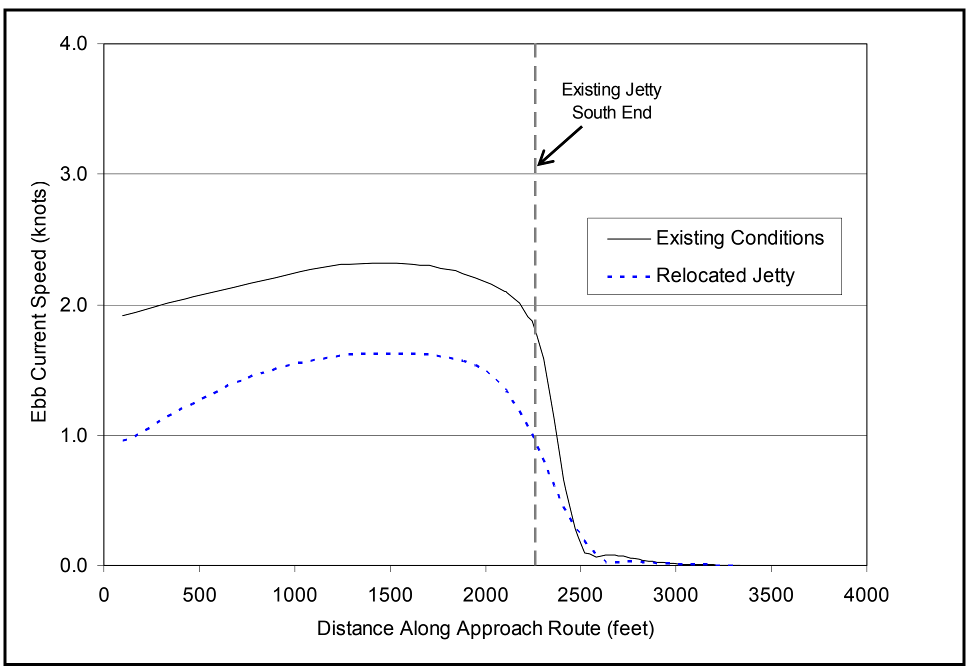

Figure 4-19 Current Speed along Navigation Channel Transect for Existing and Alternative 5 Ebb Conditions

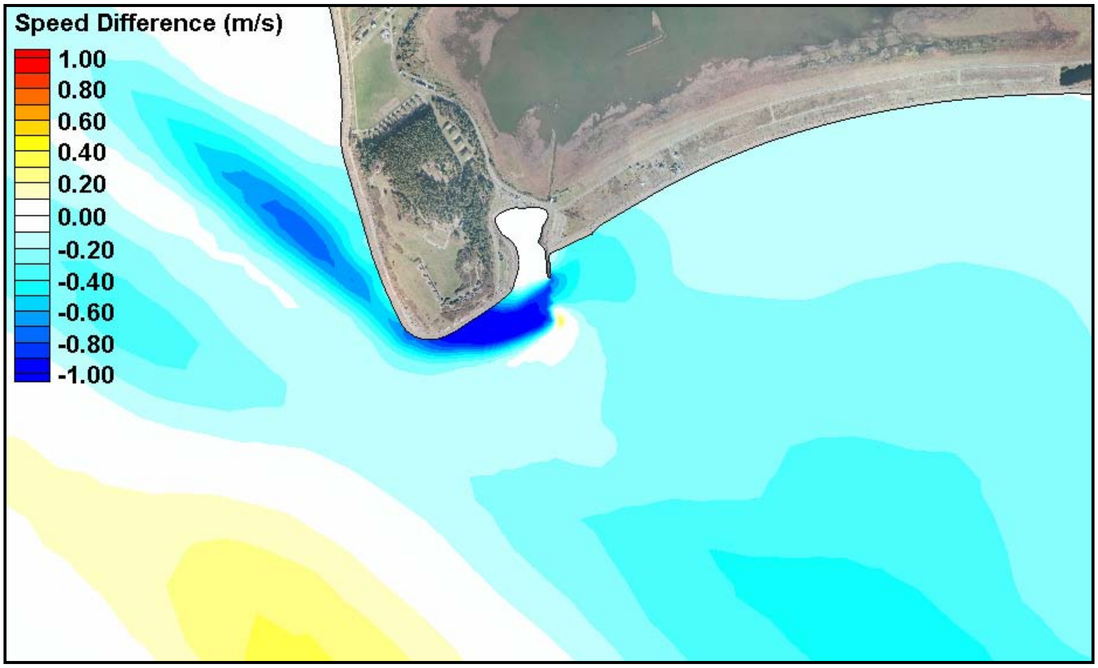

Figure 4-20 Difference in Current Speed between Alternative 1 and Existing Conditions, Ebb Currents

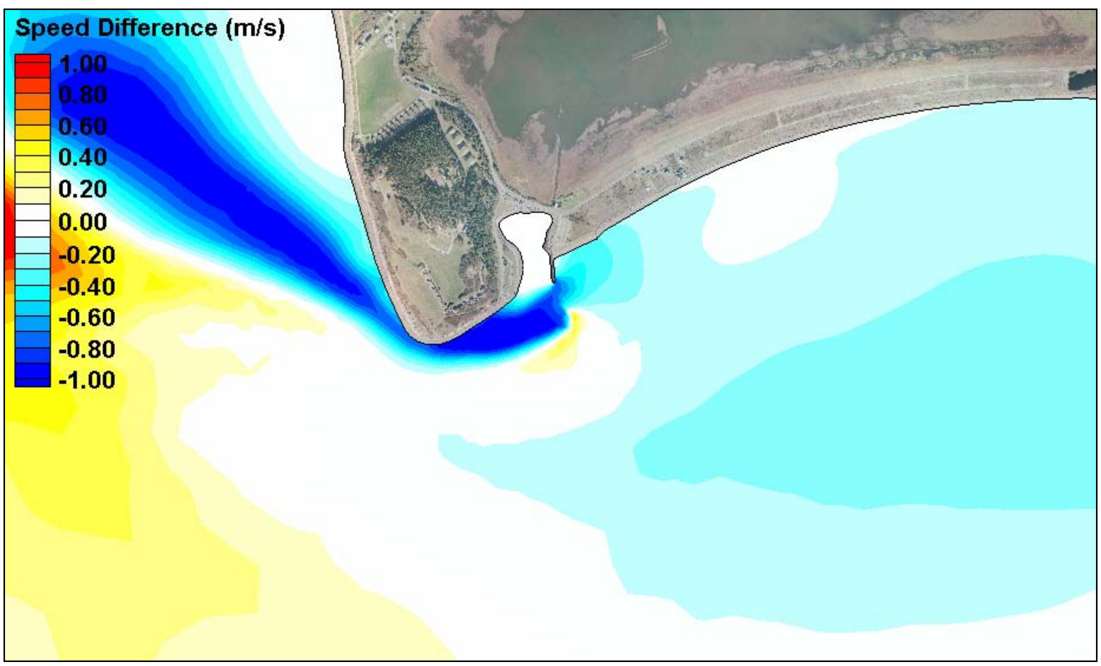

Figure 4-21 Difference in Current Speed between Alternative 3 and Existing Conditions, Ebb Currents

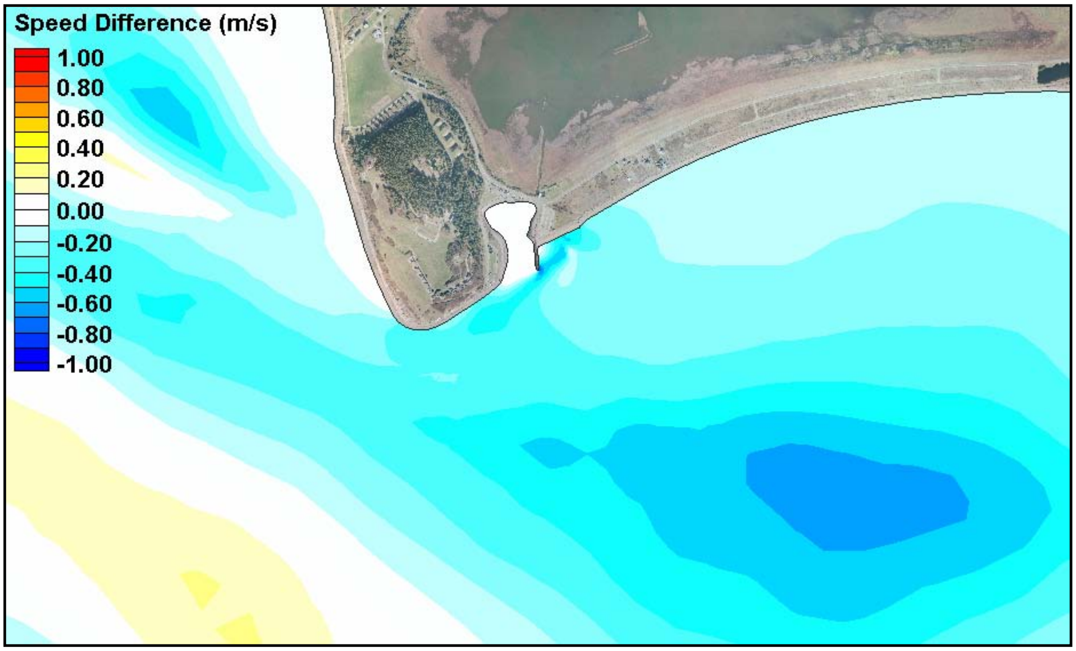

Figure 4-22 Difference in Current Speed between Alternative 5 and Existing Conditions, Ebb Currents

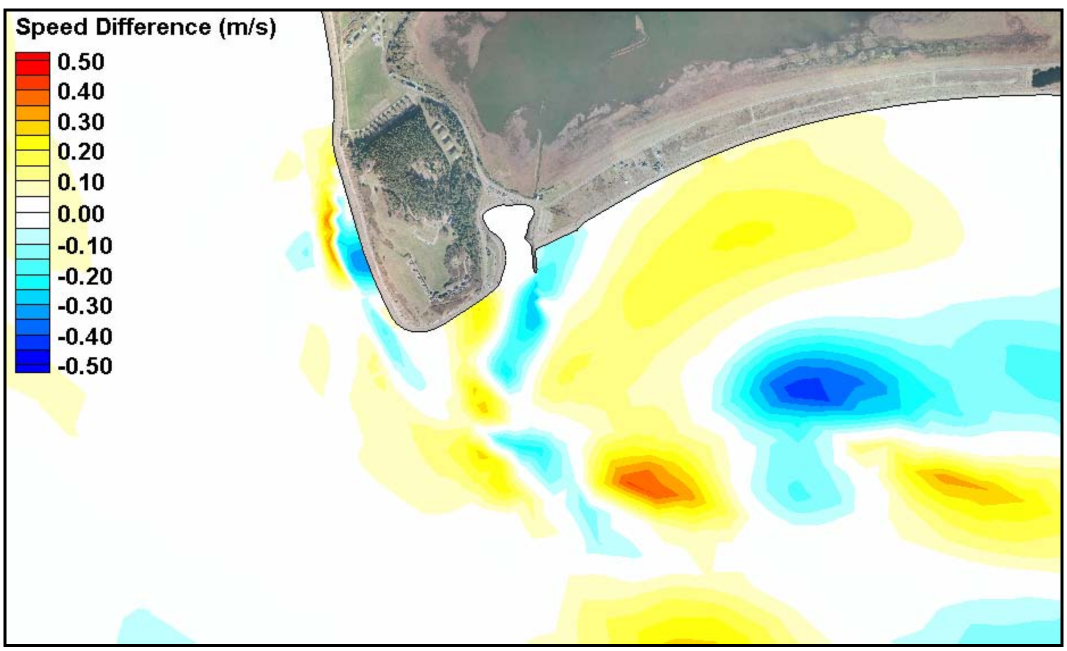

Figure 4.23 Difference in Current Speed between Alternative 1 and Existing Conditions, Flood Currents

4.2. Wave Transformation Modeling

Numerical wave modeling was undertaken to evaluate the effect of the proposed jetty alternatives on wave conditions inside of the channel and harbor. The modeling results are also were used in evaluation of potential effects from alternatives on coastal processes, specifically shoreline erosion, bottom scour, and channel sedimentation.

As discussed previously in Section 2.4, the wave climate at Keystone Harbor is complex and is comprised of several different wave systems, including local wind-waves from directions between 150 and 230 degrees (True North), wind-waves generated within the Strait of Juan de Fuca and offshore deepwater waves from the Pacific Ocean. Computer modeling was performed to investigate the complex wave conditions near the project site and potential changes to these conditions due to the proposed jetty modification alternatives. The methodology for computer modeling consisted of using two different computer models, STWAVE and COASTOX. STWAVE model was used to simulate wave transformation from offshore and wind-wave generation in the Strait of Juan de Fuca to the project site for existing conditions. COASTOX simulates local wind-generated wave transformation in the nearshore zone, including refraction, diffraction and reflection of wind-waves, for existing conditions and for the proposed jetty alternatives.

4.2.1. Models Description & Setup

4.2.1.1 STWAVE Model

The STWAVE model is a two-dimensional spectral wave transformation model that simulates wind-wave growth, refraction, shoaling, energy dissipation (due to white capping and breaking) and approximates diffraction. It should be noted that the STWAVE model does not simulate reflection and has limitation on simulation of wave diffraction processes. Therefore, the STWAVE model was not used for analysis of wave-jetty interaction and evaluation of the proposed jetty modifications alternatives.

The STWAVE model was used to investigate the wave field near Keystone Harbor constituted by waves entering from the Pacific Ocean through the Straight Juan de Fuca and determine the importance of this wave field to coastal processes and specifically shoreline erosion. STWAVE simulated waves entering from the Pacific Ocean and propagating through the Strait of Juan de Fuca, as well as concurrent wind-wave growth along the Strait.

The STWAVE modeling domain was constructed consisting of two regions; a large-area modeling domain and a small-area nested modeling domain. The large-area modeling domain extends from the open ocean through the Strait of Juan de Fuca to Admiralty Bay (Figure 4.24) with a resolution of 200 meters. Model input consisted of a directional spectrum with a certain significant wave height and peak period, as well as a constant wind speed and direction (constant in time and constant over the domain). These input conditions were taken from the La Perouse Bank Wave Buoy #C46206 (Canadian Marine Environmental Service). The buoy is located at 126.00° W and 48.83° N that is at the ocean boundary of a large-area modeling domain grid shown in Figure 4.24. It was assumed that the offshore wave spectrum could be approximated using a Jonswap spectrum with peak enhancement factor (gamma) of 3.3 and directional spreading parameter of 4.0. The system of waves that propagated through the Strait of Juan de Fuca during simulation consisted of transformed Pacific Ocean waves and superimposed wind-waves generated inside the Strait.

The small-area (nested) modeling domain covers most of the North part of Admiralty Bay with resolution 25 meters. Figure 4.25 shows the nested modeling domain with depths in color contours. Nearshore transformation processes evaluation with the small-area grid required higher resolution to accurately represent the bottom relief. The nested grid modeling was performed using wave input consisting of directional wave spectra interpolated from the large-area modeling grid results along the nested grid boundary location.

The design storm scenario for STWAVE modeling corresponds to the storm occurred on January 30th, 2004 during a CHE site visit. During this storm, offshore wind and wave information were available from the La Perouse Bank Wave Buoy. The recorded by buoy offshore significant wave height and peak period during were 5.5 meters and 10.0 seconds, respectively. The offshore wind speed was 17.0 m/sec from a direction of 270 degrees relative to True North. Based on the available statistics from La Perouse Bank Wave Buoy (Canadian Marine Environmental Service), wind speeds at this location exceed this speed approximately 3% of the time.

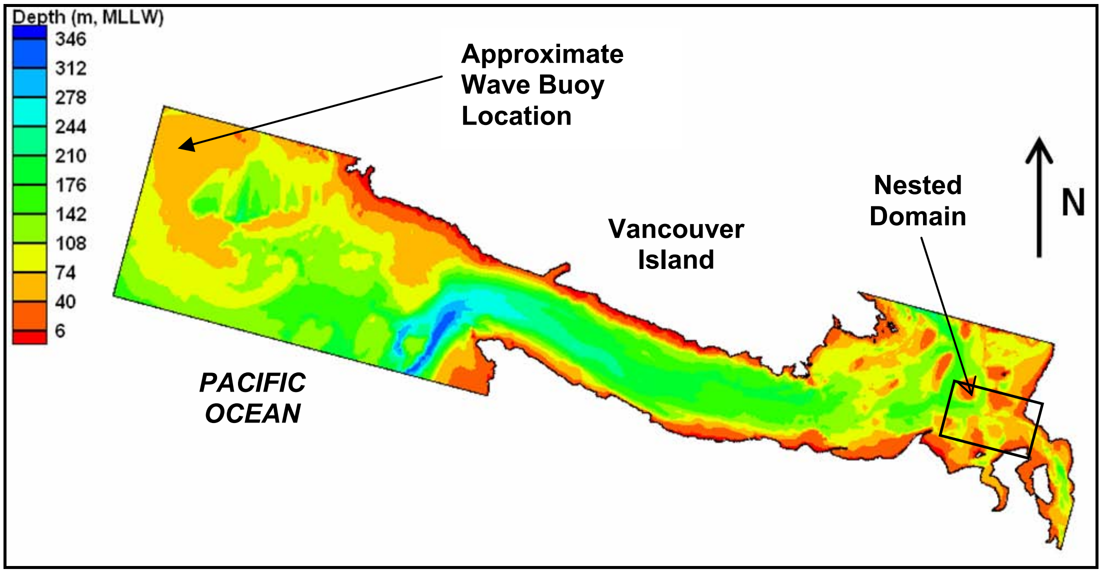

Figure 4-24 STWAVE Wide-Area Domain and Wave Buoy Location

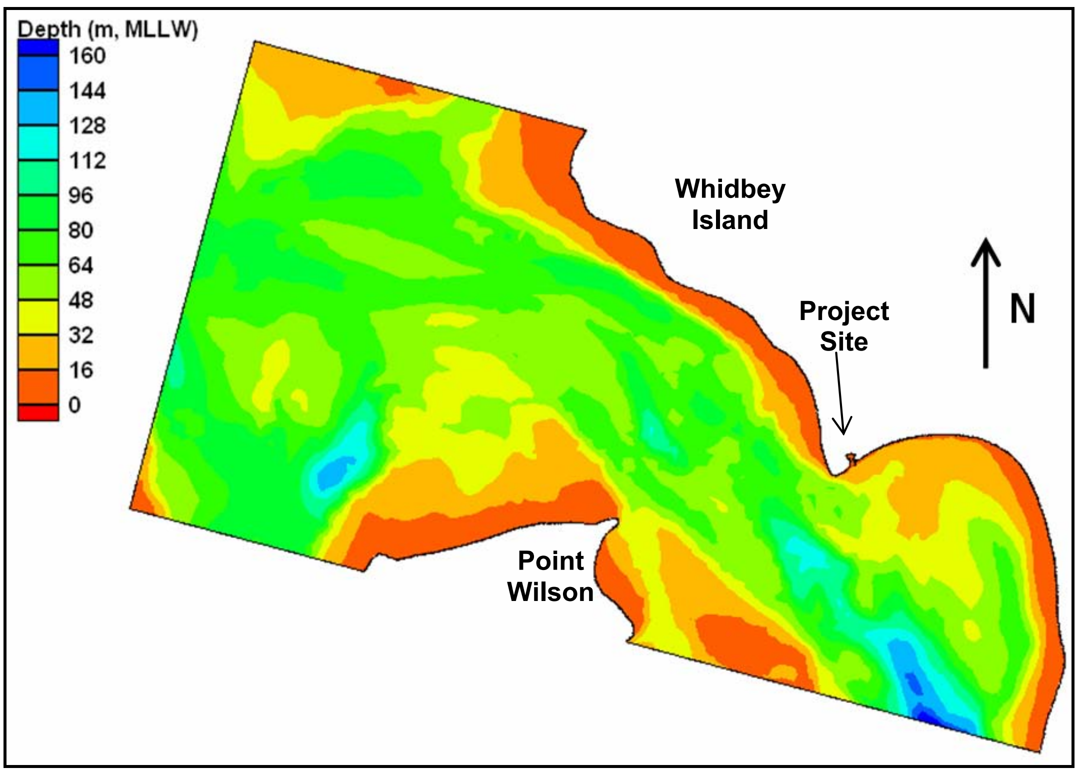

Figure 4.25 STWAVE Nested Modeling Domain

4.2.2. COASTOX Model

The COASTOX model was used to simulate the local wave transformations and interactions with the bottom slope for existing conditions and all proposed jetty alternatives. The COASTOX model is a two-dimensional monochromatic wave model that simulates refraction, diffraction and reflection on finite difference grids[6].

The COASTOX modeling domain grid was constructed using hydrographic survey data and shoreline data compiled for circulation modeling, and extends from approximately 300 m to the West of Admiralty Head to a location approximately 1km to the East of the Existing Jetty. The modeling domain grid was built with a resolution of 3.0 meters. Local wind-waves (see Section 2.3) were the basis for developing the wave input.

Waves from two directions, Southwest and Southeast, were considered during the modeling procedure. The design storm conditions for both directions were estimated at wave height and period of approximately 1.15 meters and 3.9 seconds, respectively. These waves were generated by a sustained wind 40 miles per hour (17.0 m/sec) which corresponds to an approximately 2-year event from this direction. All wind-wave transformation modeling was performed at a tidal elevation of Mean Sea Level.

4.2.3. STWAVE Modeling and Results

STWAVE large-area modeling results for the observed wave storm are shown in Figure 4-26. This figure shows the distribution of significant wave heights over the modeling domain in color contours. The model shows a decrease in significant wave height as the waves travel through the Strait of Juan de Fuca, from 5.5 meters at the offshore boundary to less than 2.0 meters at the nested grid boundary. It should be noted that the decrease in wave energy is large in spite of the fact that additional wind energy was input into the wave spectrum during propagation through the Strait. The spectrum is gradually changing from the offshore input spectrum, whose energy is at longer periods and is being dissipated in refraction and other processes, to a more local wind-wave spectrum generated in the Strait of Juan de Fuca.

Figure 4-27 shows significant wave heights and peak wave directions over the nested domain. The STWAVE model shows further reduction of wave heights during propagation to the project site. Wave height is reduced from approximately 2.0 meters at the boundary of the nested grid to

[6] It should be noted that existing wave spectrum models do not provide accurate simulations of wave diffraction and reflection processes. Therefore the monochromatic COASTOX model that proved to be an accurate tool in evaluation of these (diffraction and reflection) processes was selected as the appropriate modeling tool.

approximately 0.2 meter and less at the seaward end of the existing jetty. The results of the modeling indicate that even during large storm events directed along the Strait of Juan de Fuca into Admiralty Bay, wave heights at the Keystone Harbor entrance are relatively small due to energy dissipation during the process of refracting nearly 90 degrees around Admiralty Head.

The STWAVE model predicts a significant wave height and peak wave period of approximately 0.2 meters and 9.1 seconds, respectively, at the tip of the existing jetty. The peak period in Admiralty Inlet is approximately 6.9 seconds, indicating that the longer-period energy in the spectrum refracts around Admiralty Head more effectively, shifting the peak period of the spectrum to 9.1 seconds. The practice of coastal engineering shows that these types of waves with small height and longer periods typically do not cause adverse impacts to the shoreline and are not of any significant concern with regard to erosion processes. To reduce the complexity in evaluation of the alternatives, the STWAVE model was not used for evaluation of jetty extension alternatives. Evaluation of jetty extension alternatives was conducted based on only COASTOX model results.

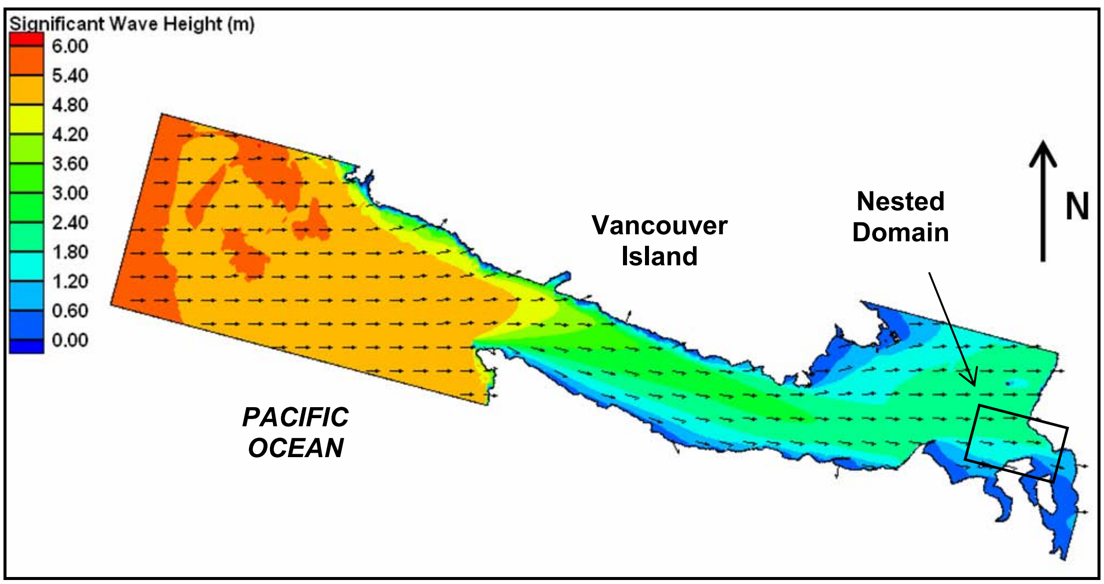

Figure 4.26 Significant Wave Heights and Peak Wave Direction, Wide-Area Domain

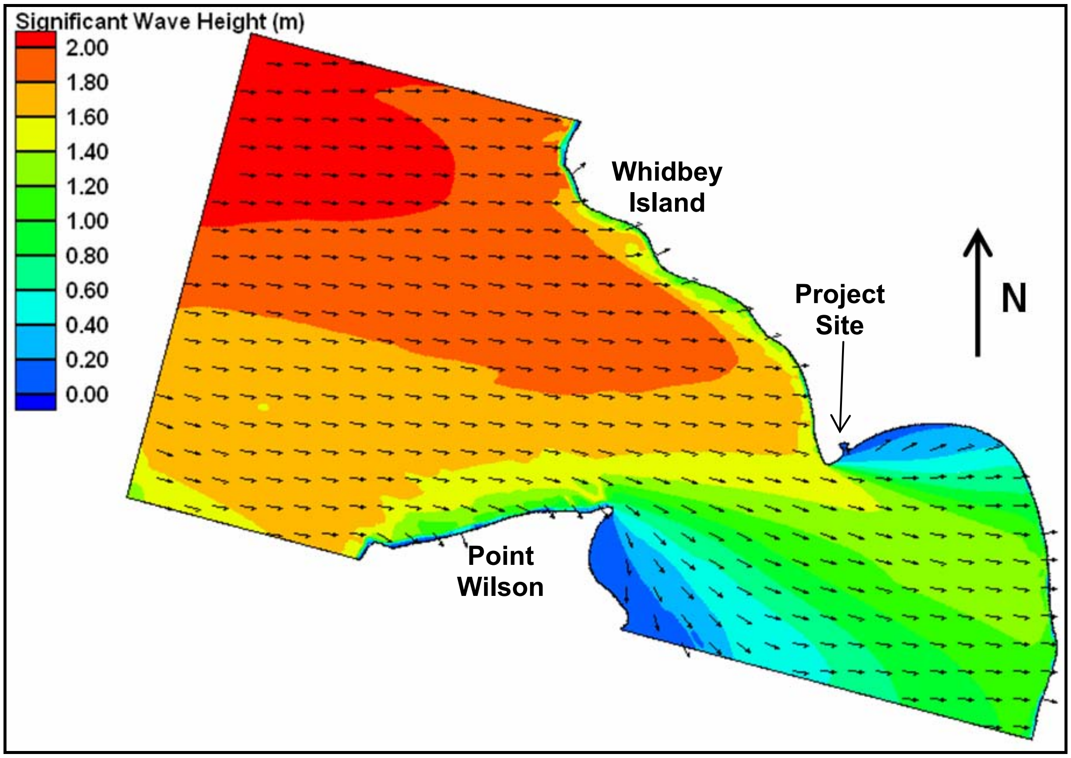

Figure 4.27 Significant Wave Height and Peak Wave Direction, Nested Domain

4.2.4. COASTOX Modeling and Results

The COASTOX model was used to simulate wave refraction, diffraction and reflection for the existing conditions and all jetty extension alternatives. Three methods of evaluating the modeling results were used in the study: Method 1 is based on analysis of spatial over the modeling domain (plan-view) distribution of wave heights, Method 2 is based on analysis of wave heights extracted along the centerline of the channel, and Method 3 is based on analysis of wave heights extracted along the same depth contour-line.

Method 1 analysis included plotting the results of modeling as spatial distribution of wave heights over the modeling domain and calculating differences in wave heights between existing conditions and appropriate alternative for each element of the modeling grid. Figures 4.28 and 4.29 show spatial distributions of wave heights over the project area for existing conditions during SW and SE storm events. Figures 4.30 and 4.31 show spatial distributions of wave heights over the project area for Alternative 5 during SW and SE storm events. Figure 4.32 shows spatial distributions of wave heights over the project area for Alternative 3 during SE storm event.

The plots with changes of wave height distributions for Alternatives 3 and 5 (relative to existing conditions) are shown at the next set of figures. Figure 4.34 and 4.35 show the differences in wave heights between

Alternative 5 and existing conditions for wave storms approaching from SW and SE respectively. Figure 4.33 shows the differences in wave heights between Alternative 3 and existing conditions for storms approaching from SE.

Method 1 wave modeling analysis shows the following major changes in wave height distributions for tested alternatives relative to existing conditions for both wave storms, from SW and SE.

- Reductions of wave heights would occur at the east side of the jetty for Alternatives 3 and 5 during storm from SW.
- Reduction of wave heights would be observed also for Alternative 3 on the west side of the jetty during storm from SE.
- Alternative 5 would increase wave heights in the harbor for both storm conditions, SE and SW. The increase of wave height is a result of deepening and widening the channel. Maximum increase of wave heights would occur during SW storms and will be in a range of 1-2 feet during the extreme storm conditions.

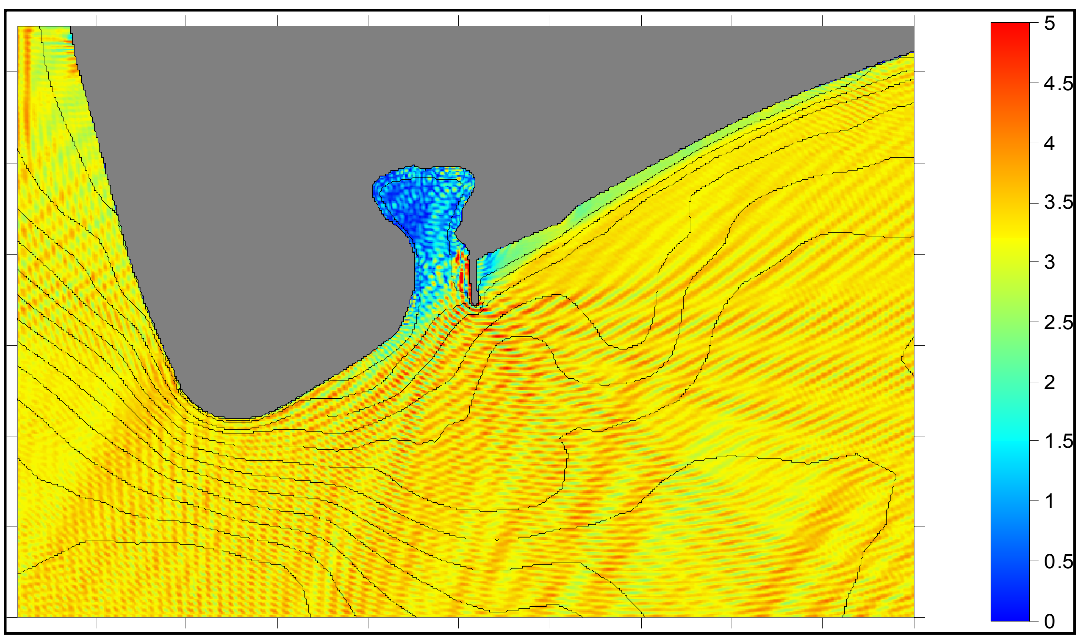

Figure 4.28 Method 1 of analysis, Existing Conditions – Southwest Waves, H=3.3 ft T=4 s.

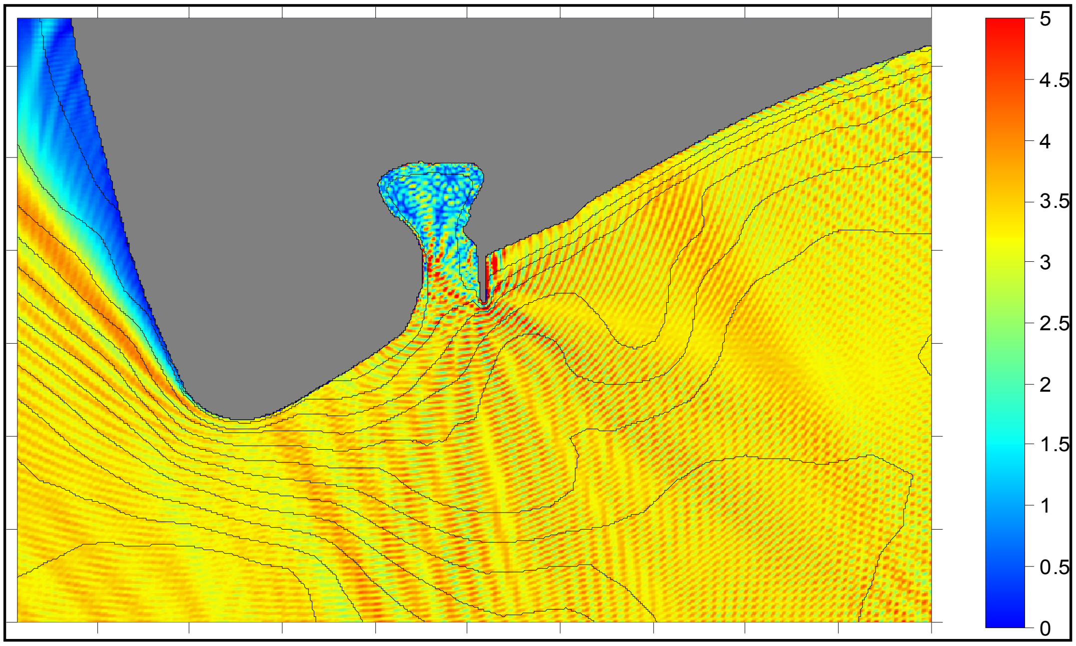

Figure 4.29 Method 1 of analysis, Existing Conditions – Southeast Waves, H=3.3 ft T=4 s

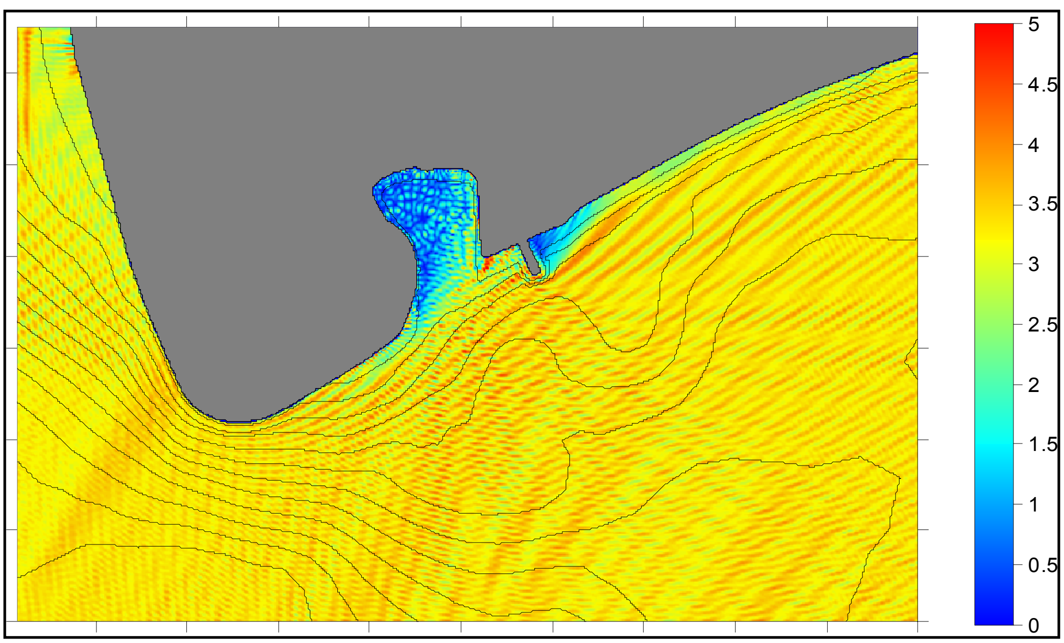

Figure 4.30 Method 1 of analysis, Alternative 5, Southwest Waves, H=3.3 ft T=4 s

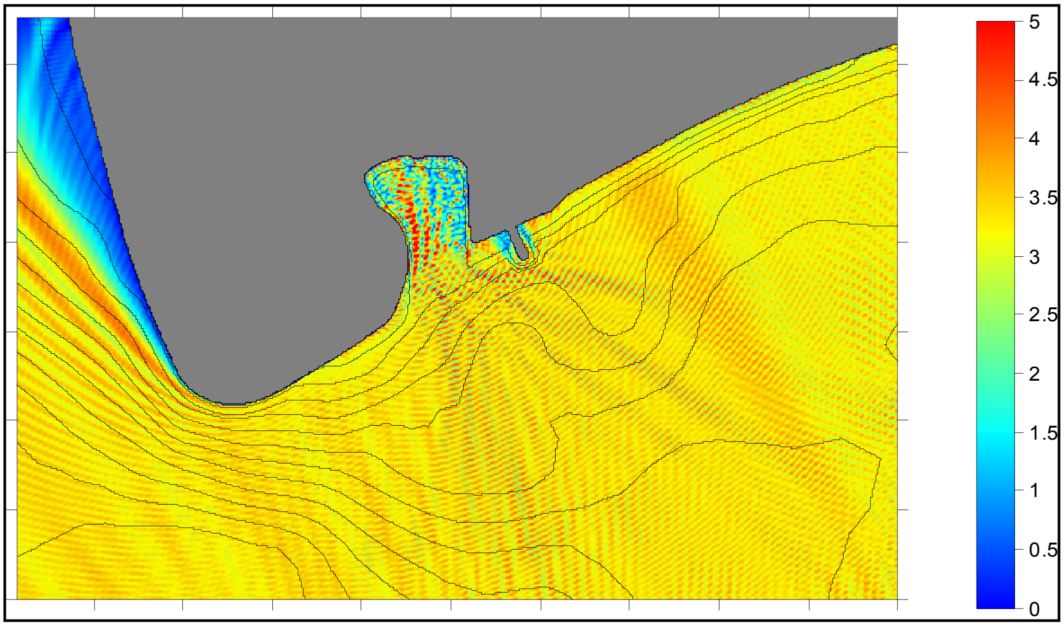

Figure 4.31 Method 1 of analysis, Alternative 5, Southeast Waves, H=3.3 ft T=4 s

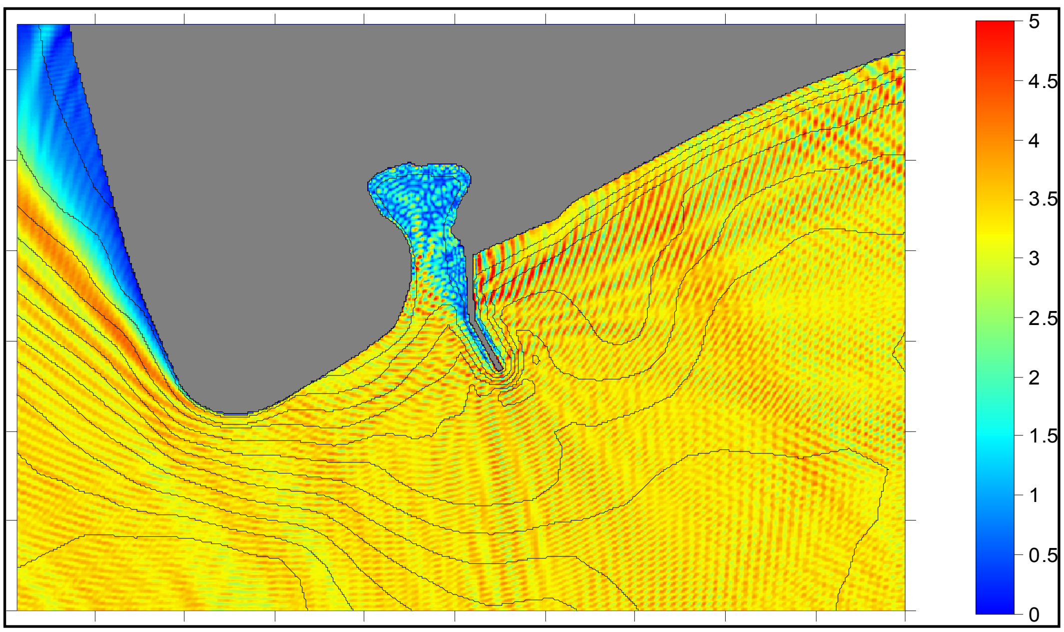

Figure 4.32 Method 1 of analysis, Alternative 3, Southeast Waves, H=3.3 ft T=4 s

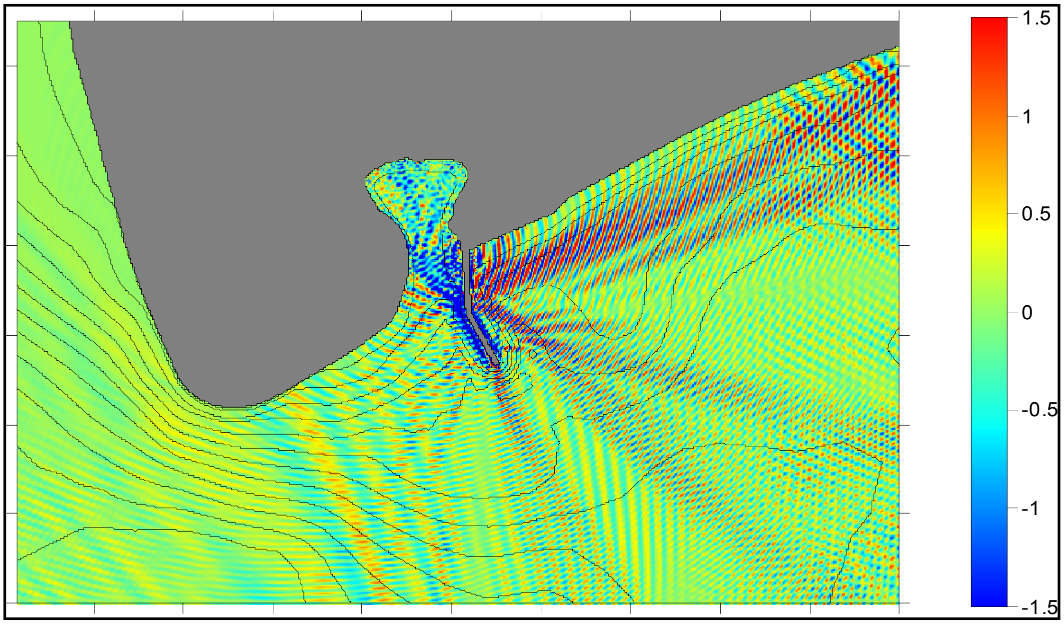

Figure 4.33 Method 1 of analysis, Alternative 3 “minus” Existing Conditions, Southeast Waves

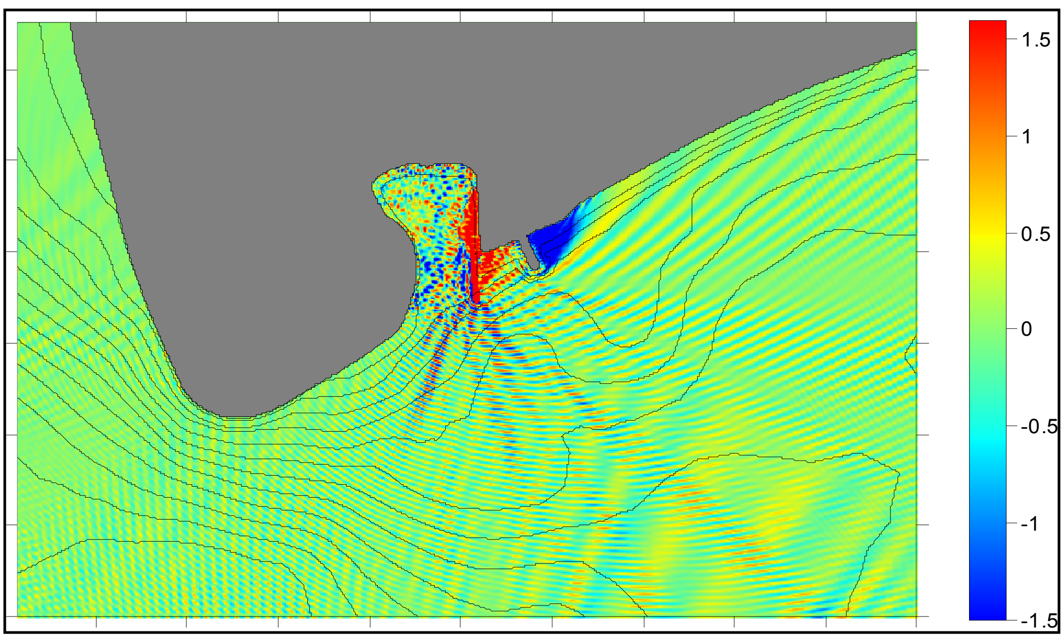

Figure 4.34 Method 1 of analysis, Alternative 5 “minus” Existing Conditions, Southwest Waves

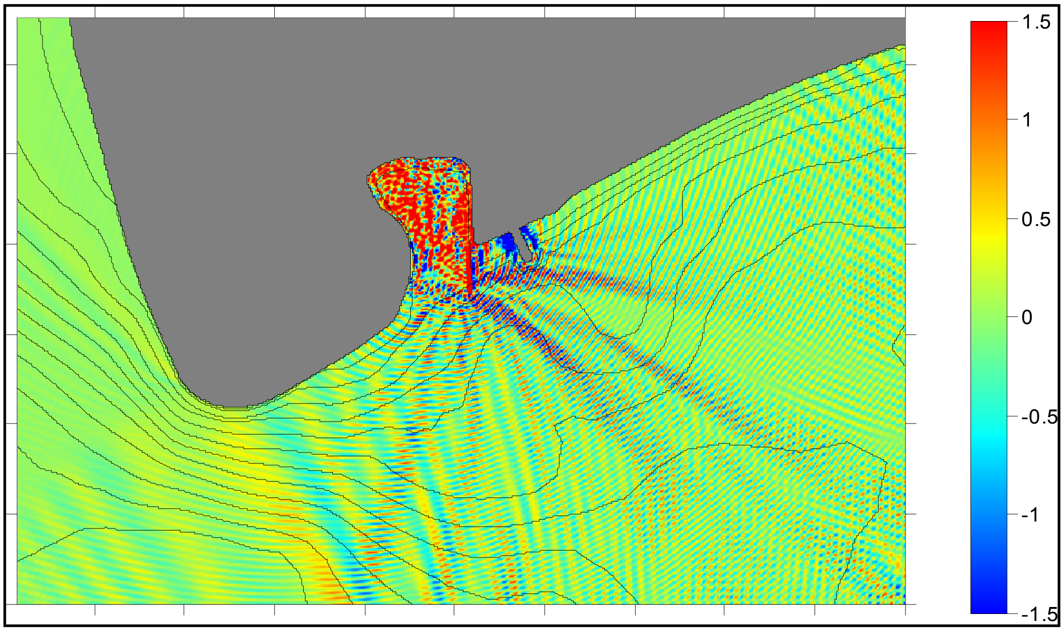

Figure 4.35 Method 1 of analysis, Alternative 5 "minus" Existing Conditions, Southeast Waves

Method 2 analysis includes extracting wave heights along the centerline transect of navigation channel for No Action (existing conditions) and other alternatives and comparison of them (wave heights). Location of the centerline transect is similar to that that shown above in Figure 4.16. The importance of that analysis is to determine the effect of jetty extension and relocation alternatives on navigation conditions in the channel. It is understood that none of the analyzed alternatives would alter wave conditions in the channel if waves are coming from SW. Therefore, the analysis using Method 2 was conducted for those waves that are approaching from SE.

Figure 4.36 shows the wave heights along the channel centerline for existing conditions and for Alternative 3 and Figure 4.37 – for existing conditions and Alternative 5. Modeling wave storm conditions are: Direction- from SE, Wave height, H=3.3 ft, wave period, T =4.0 seconds. It is noted that Alternative 3 reduces wave heights in the channel along the extended jetty and does not change existing wave conditions in the harbor. It should be also noted that Alternative 3 modeling scenario does not include deepening and widening the channel.

Similar to the observation from Method 1 analysis, Figure 4.37 shows that relocation of the jetty (Alternative 5) would not change wave in the channel in any significance, but would increase wave heights in the harbor relatively to existing conditions. Maximum increase of wave height during modeling storm conditions is estimated at approximately 2.0 ft. Increase of wave heights in the harbor is a result mostly of deepening and widening the channel that allows more wave energy penetrate to the harbor.

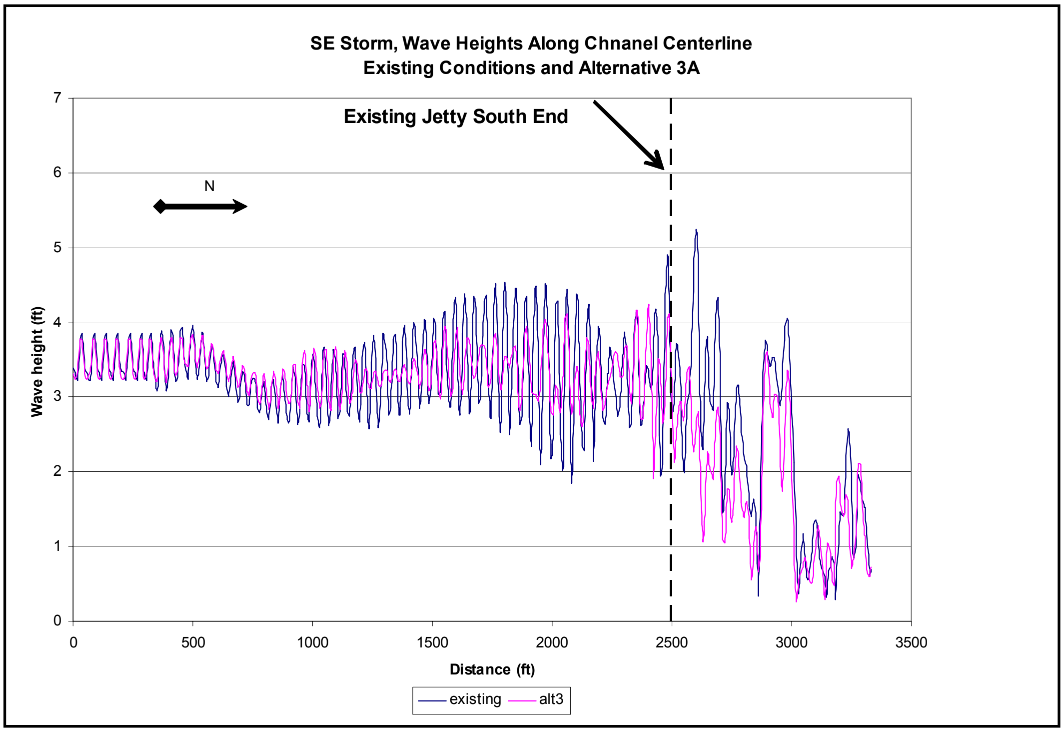

Figure 4.36 Method 2 analysis, Wave heights along channel centerline. Existing Conditions and Alternative 3, SE storm H=3.3 ft, T= 4,0 seconds

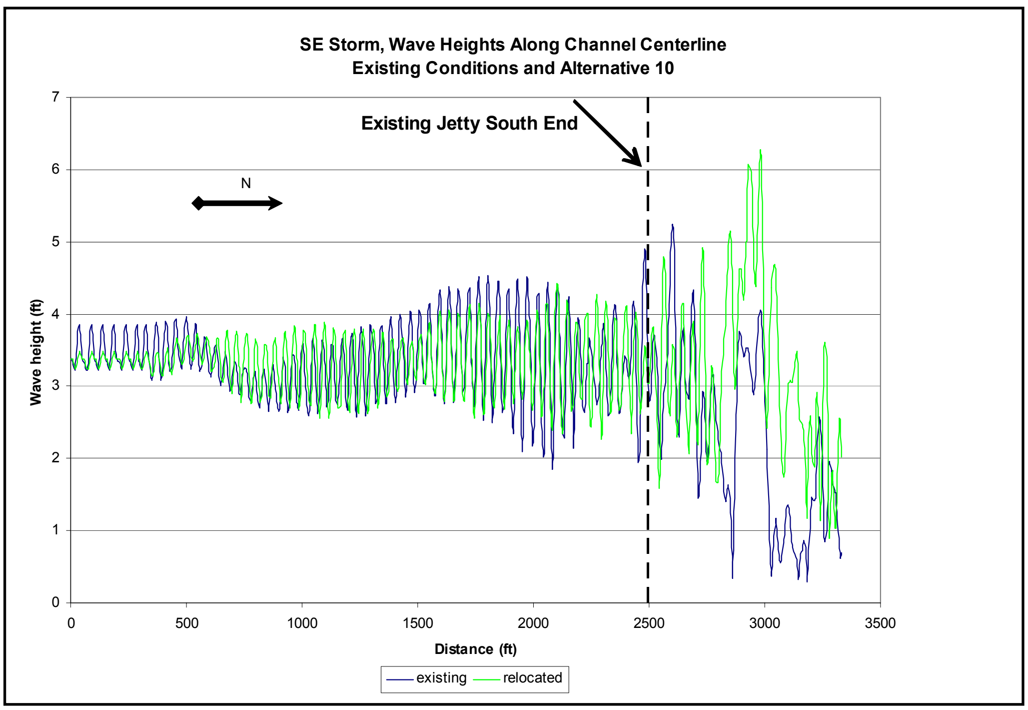

Figure 4.37 Method 2 analysis, Wave heights along channel centerline. Existing Conditions and Alternative 5, SE storm H=3.3 ft, T= 4,0 seconds

Method 3 analysis includes extracting wave heights along 3-meter (10 ft) depth contour-line from east and west sides of the entrance to the harbor. Location of the contour-lines is shown in Figure 4.38. The importance of Method 3 analysis was in determining the effect from the jetty extension alternative on shoreline erosion mostly to the west from the jetty. Understanding that none of the jetty extension and relocation alternatives would adversely effect on wave climate to the west from the harbor entrance, Method 3 analysis was conducted only for the waves approaching from SW.

Wave heights measured on the model along the contour-line for all jetty extension alternatives and existing conditions are shown in Figure 4.39. Modeling wave storm conditions are: Direction- from SE, Wave height, H=3.3 ft, wave period, T =4.0 seconds. The analysis of the figure shows that most of jetty modification alternatives would reduce wave heights along the shoreline relatively to existing conditions during wave storms approaching from SW. The reduction of wave heights is due to a shading effect from the extended jetty. This shading effect would diminish to the east of the jetty and no changes to wave conditions (relatively to existing conditions) would be expected at approximately 2,000-3,000 ft eastward of the existing jetty. t.

Alternative 3B, however would result in different and more complex wave height increase-reduce pattern along the shoreline. This complexity results from submerged extension of the jetty and may not be desirable from the perspective of shoreline stability.

Alternative 5, jetty relocation alternative would increase wave heights along the shoreline of approximately 300 ft that would be exposed to wave impact after jetty relocation. Further to the east, wave heights would be reduce to the shag effect from the relocated jetty.

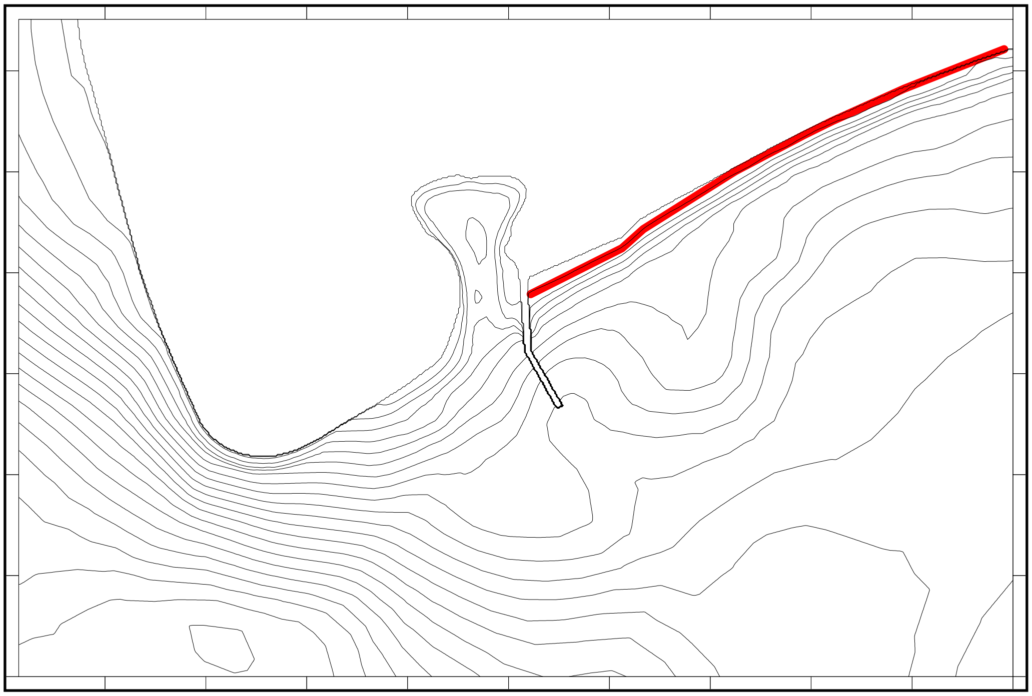

Figure 4.38 Method 3 analysis, 10 ft depth contour-line

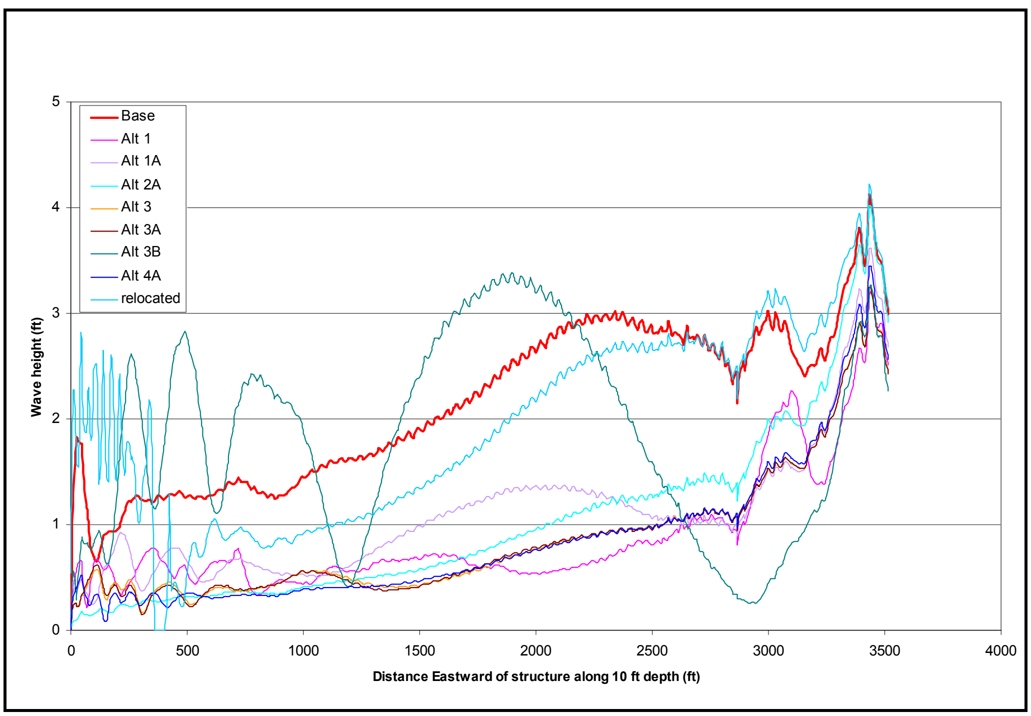

Figure 4.39 Method 3 analysis, Wave heights along 10 ft contour-line, SW storm, H=3.3 ft, T=4.0 seconds

Summarizing the results of analysis using three methods, the following have been concluded:

- All jetty extension alternatives would reduce wave heights approaching from SE in the channel located along the extended jetty. All jetty extension alternatives (if no deepening and widening channel occurs) would not alter wave conditions in the harbor relatively to existing conditions.
- Relocation of the jetty (Alternative 5) would not change waves in the channel at any significance that may effect the navigation for storms approaching from SE and SW
- Relocation of the jetty (Alternative 5) would increase wave heights inside of the harbor relative to existing conditions for both storm conditions, SE and SW. Maximum increase of wave height during extreme storm event, approaching from SE is estimated at approximately 2.0 ft. Increase of wave heights in the harbor is a result mostly of deepening and widening the channel that allows more wave energy penetrate to the harbor
- All jetty extension alternatives except Alternative 3B would significantly reduce wave energy along the shoreline during wave storms approaching from SW.
- Relocation of the jetty (Alternative 5) would reduce wave energy along the shoreline during wave storms approaching from SW
- In the cases of both rock jetties and vertical wall barriers, the composite-alignment solutions seem to provide better protection in the lee side, as they are oriented almost normal to the incident storm wave direction and create significant shadow areas behind the structure.

4.3. Sediment Transport, Sedimentation/Scouring Modeling

The objective of sediment transport modeling was to evaluate the proposed alternatives from the perspective of potential impact on sedimentation in the navigation channel and effects on scouring or accretion in areas adjacent to the jetty. Review and analysis of general sediment transport at Keystone Harbor presented in Section 2 of the report showed the follows:

- No effect on eastward (from west to east) sediment transport should be expected from any jetty configuration alternatives because the existing jetty has been already a complete blockage to the littoral drift from west to east.
- Medium and fine sand and smaller particle of sediment may be transported from the east site of the jetty to the west site and accumulated in the channel. This sediment transport mostly is driven by

tidal flow. The jetty alternatives may effect the westward (from east to west) sediment transport and channel sedimentation rates.

Based on these findings, in order to compare alternatives, the computer model was selected and setup to simulate tidal flow sediment transport in the project area as described in the following sections.

4.3.1. Model Description

Sediment transport at Keystone Harbor was evaluated using the LAGRSED two-dimensional sediment transport model. The basis of selecting this model was its unique ability to simultaneously simulate transport of different types of sediment that constitute the project littoral system. Other sediment transport models are available in the industry but typically simulate transport of one sediment grain size at a time.

The LAGRSED model has been developed by Coast & Harbor Engineering in cooperation with scientists overseas under a federal grant from U.S. Civilian Research and Development Foundation (CRDF). The LAGRSED model is a finite-element, Lagrangian sediment transport model that simulates current and wave-induced transport of non-cohesive (sand) and cohesive (silts and clays) sediment and mixtures of sediments (Maderich *et al*, in print). The model employs multiple formulations to calculate time variation of suspended sediment concentrations, suspended transport, bedload transport, bed elevation change and armoring. The LAGRSED simulations for Keystone project were run using only input tidal currents from the ADCIRC model. It should be noted that the results of LAGRSED modeling are used in this study only for comparative analysis of the alternatives in a qualitative manner due to lack of model calibration data. Therefore, it is likely that uncertainties that may occur due to the use of only tidal currents will not significantly affect the conclusions of the analysis.

4.3.2. Model Domain Setup

As discussed above, the LAGRSED modeling results are used herein in qualitative manner for comparison analysis only. No field data were available to validate the model and quantitatively evaluate the model performance (these data are typically not required for this type of analysis). Model boundary conditions were identical for all alternative simulations and therefore allow consistent comparison of the effects of each alternative relative to existing conditions.

The sediment transport (LAGRSED) modeling domain was built to encompass the areas of interest, including the navigation channel near the jetty and shoreline. The sediment transport model was run using a small-scale domain to enhance model efficiency since simulation of multiple sediment types requires a large amount of computation. Model bathymetry data were taken from the large-scale ADCIRC domain discussed in Section 3.3. Figure 4-40 shows the LAGRSED model domain for existing conditions.

The variable input parameters into the models were current velocities and tide elevations from ADCIRC model and sediment characteristics from the modeling area. The total ten day simulation ADCIR model run was used to develop input hydrodynamics for LAGRSED[7]. Results from this simulation were used for analysis of LAGRSED results to evaluate the change of the bed elevations and calculate erosion/accretion rates. The period of simulation represents typical tidal flow conditions for Keystone Harbor area and corresponds to the range of tidal fluctuation observed from February 28 to March 9, 2004. ADCIRC model results (water levels, velocities) from the large-scale finite element domain were interpolated onto the small-scale LAGRSED domain using custom finite element mapping routines.

Sediment characteristics used as input into the LAGRSED model were developed based on information available from various sources, including: Coast & Harbor Engineering (CHE) site visits, Corps of Engineers boring logs from dredging reports, and the Keystone Terminal Relocation Feasibility Study (CH2MHill 2003). These data sources indicated the presence of various types of material around the project area and were used to develop a spatial distribution of sediment size for input into the modeling. Figure 4-41 shows a site aerial photo and notes the sediment sizes used for the modeling in each area. The sediment characteristic information was digitized and incorporated into a spatially variable grid of input sediment sizes.

Figure 4-42 shows the sediment size distribution grid used for input in sediment transport modeling. It should be noted that the sediment in the vicinity of the jetty, including the West and East sides, were assumed to be sand in the model. In reality, the bottom in this area consists of a wide range of material including sand/gravel/cobble, and the composition of the bottom is complicated by dredged material placed periodically in this area by the Corps of Engineers. CHE site visits determined that the visible sand material (assumed to have median diameter 0.3mm) is concentrated near the jetty and at the lower parts of the beach (see Figure 4.43 and 4.44).

The source of the sand in the channel is likely tidally driven sediment flux in Admiralty Bay and sediment washed away from the dredged material placement by the Corps of Engineers (see Figure 2.24 from Section 2.6). Based on observation and preliminary analysis it is suggested that input sediment in the model in bottom areas near the jetty is best represented as

[7] Please note that only modeling results of seven day simulation were used for the analysis.

sand. This assumption allows evaluation of the alternatives from the perspective of their ability to maintain the sandy beach and reduce sand transport to the channel.

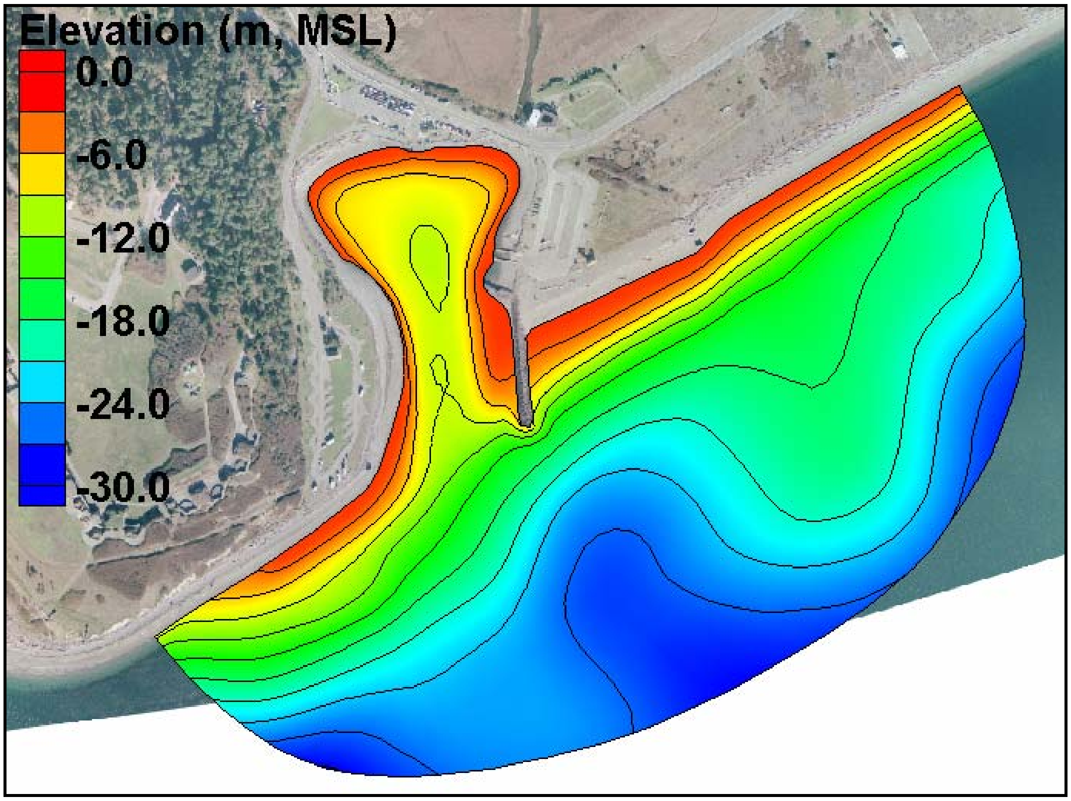

Figure 4.40 LAGRSED Model Domain for Existing Conditions

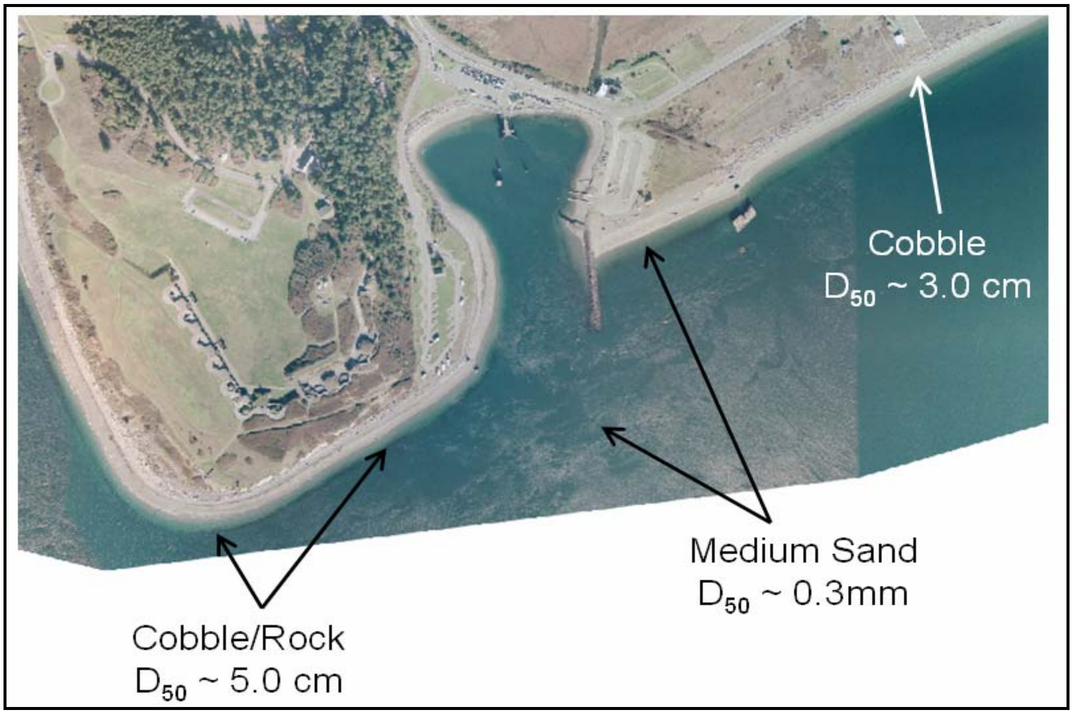

Figure 4.41 Site Aerial Photo and Sediment Sizes used for Sediment Transport Modeling

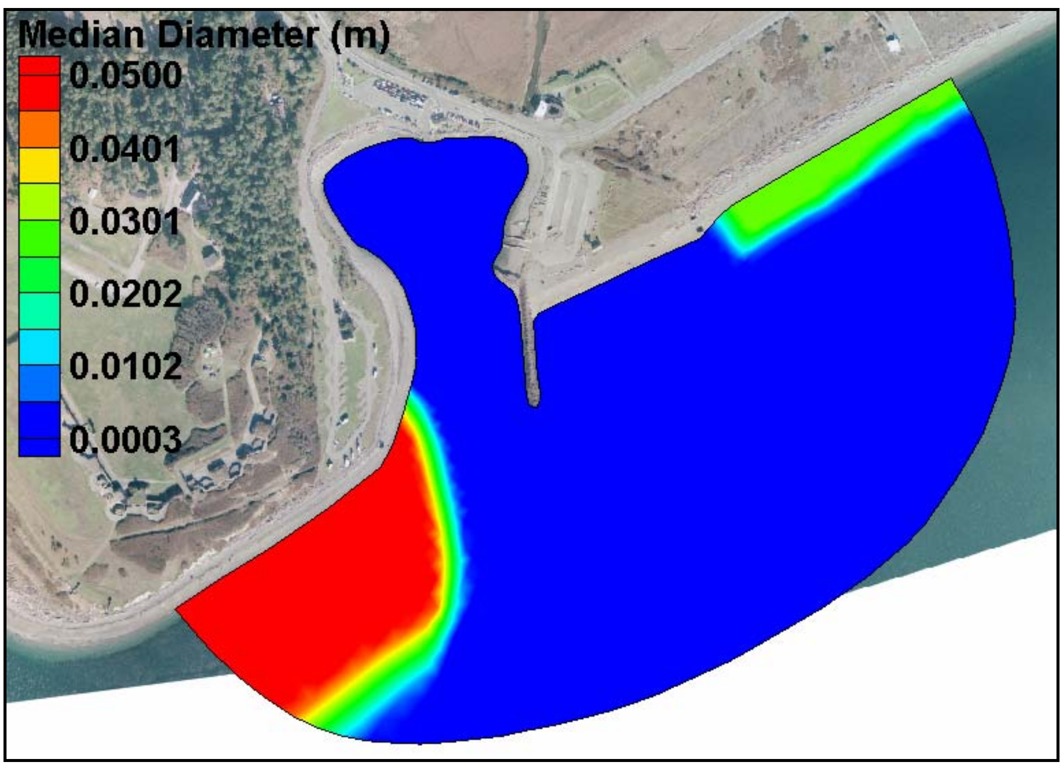

Figure 4.42 LAGRSED Model Input Sediment Size Distribution Grid (note that sediment sizes on the legend are in meters and represent the sediment median diameter)

Figure 4.43 Sand Material Observed in the Lower Beach

Figure 4.44 Sand Material Observed at the Jetty in Dredged Material Placement Area

4.3.3. LAGRSED Model Results

4.3.3.1 LARGSED Modeling Results for Existing Conditions

Figure 4-45 shows the results of LAGRSED modeling that is a bottom change (erosion and accretion) after seven days of simulation for existing conditions. The bottom changes are represented by color contours. The red/brown indicate accretion, yellow indicates no significant changes and the cyan/blue indicate erosion. Review of the existing conditions modeling results indicates the following:

- Sand accumulates on the east side of the jetty under tidal currents. This result corresponds to field observations and discussion above regarding the sandy beach in the vicinity of the jetty.
- Sand erodes on the east side of the dredged material placement area. The model predictions in this location appear to be reasonable based on predicted tidal currents. Figure 4.46 is a photograph taken at the location assumed to be sand in the model and where erosion is predicted by LAGRSED. The figure shows no significant amounts of sand in this area. The observed beach material is composed of gravel and cobbles.
- Sediment accumulates in the navigation channel. This result corresponds to information from the Corps of Engineers regarding dredging areas. In addition, the shape of sedimentation (underwater spit) is typical for patterns of sedimentation observed at other navigation projects with similar jetties.
- No significant bottom changes on the West side of the navigation channel. Although no data were available to verify this result, it appears reasonable because of the large size of bottom material and strong tidal currents observed in this area.
- Erosion occurs in deep water seaward of the harbor entrance. This result appears to be consistent with the general bathymetry of the area. The bottom depression located seaward of the jetty was observed in all available hydrographic surveys. Though this depression is slightly offset from the entrance in the Eastward direction, the general trend of scour shown in the model (depth of scour and orientation of the scour) indicates a similarity between the modeling results and typically observed field processes.

Considering the above discussion, the LAGRSED model results should be considered reasonable for the project area from the perspective of qualitative analysis and comparison of the alternatives.

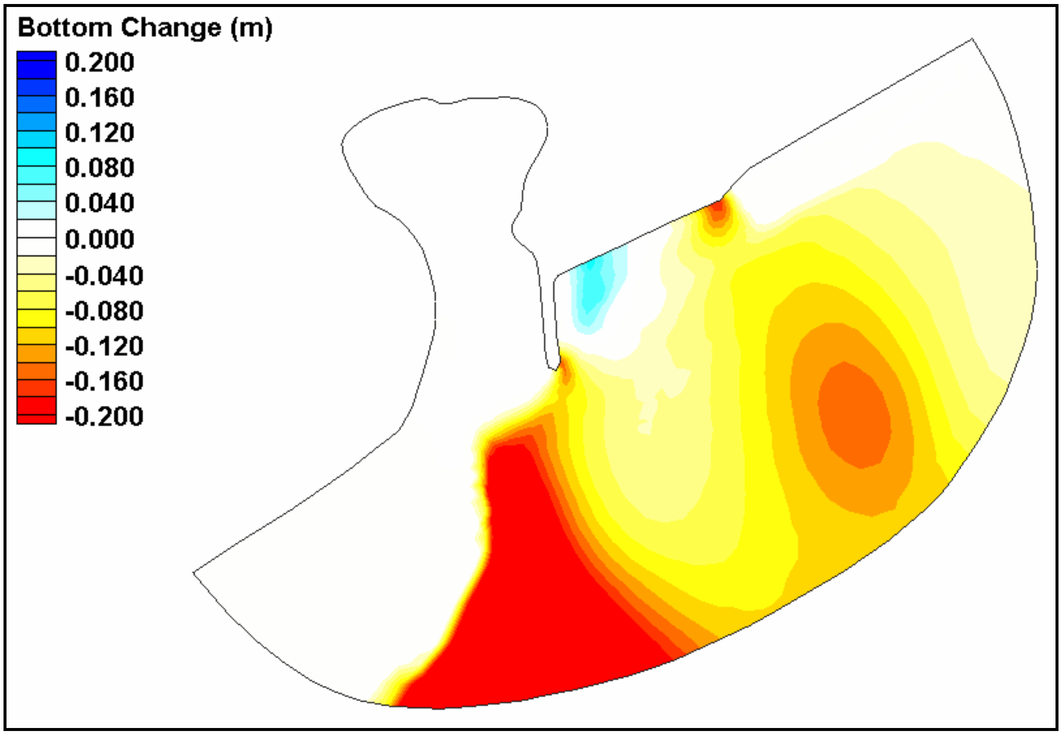

Figure 4.45 LAGRSED Model Existing Conditions Bottom Elevation Change after Seven-Day Simulation

Figure 4.46 East Beach near Erosion Area in LAGRSED Modeling Results

4.3.3.2 LARGSED Modeling Results for Project Alternatives

Each of the project alternatives was constructed on corresponding modeling grids and LAGRSED modeling was repeated for the same input parameters as existing conditions. Figure 4.47 shows seven-day bottom change for Alternative 1 as an example. Comparison between the alternatives was conducted with regard to two qualitative criteria:

- Channel sedimentation. Evaluation was conducted to determine if the alternative may increase sedimentation in the channel footprint (Figure 4.48). The increase of the sedimentation may results in increase maintenance dredging requirements that is a negative consequent for the project.
- Shoreline effect. Evaluation was conducted to determine if scour of the bottom slope and beach area would increase relatively to the existing conditions. If scour is observed, the erosion of the shoreline is assumed that is a negative consequent for the project.

The comparison is summarized in Table 4.8.

Table 4.8 Alternative Qualitative Comparison Based

Alternatives	Channel Sedimentation	Shoreline Effect
1	No increase	No effect
1A	No increase	No effect
2	Significant increase[8]	Small Effect
2A	No increase	No effect
3	No increase	No effect
3A	No increase	No effect
3B	Small increase	Insignificant effect
4	Increase	Insignificant effect
5	Short term increase during stabilization period	No effect to the east

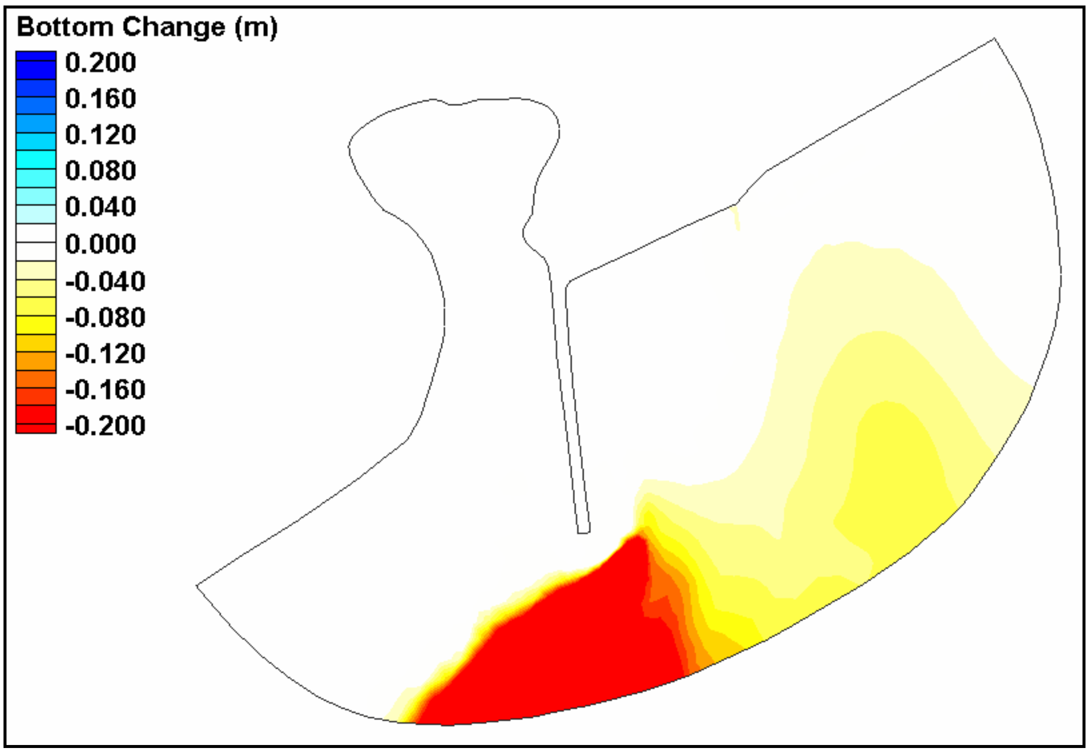

Figure 4.47 LAGRSED Model Alternative 1 Bottom Elevation Change after 10-Day Simulation

[8] Modeling result shows that significant amount of sediment would deposit immediately at the westward side of the jetty. This is because of clearance at the bottom of the jetty dramatically increases bottom velocities and abruptly drops them just after the jetty. It is believed that in real conditions the sediment would transport farther to the west and accretes in the channel area.

Figure 4.48 Existing Navigation Channel Footprint and Sedimentation Evaluation Area

Summarizing the results of sediment transport modeling and considering the study of sediment transport discussed in Section 2 the following has been concluded:

- In general, because of a significant predominance of gravel-coarse sand sediment transport from west to east no significant changes to sedimentation in the channel may occur for any of the alternatives consisting of full cross section jetty extension or modifications at the east side of the channel. This conclusion is not applicable to Alternative 2 that includes open space at the bottom of the jetty (see below).
- Jetty relocation alternative may result in short-term increase sedimentation in the channel to widening and deepening. A new dredged cut stabilization would occur that usually increases sedimentation in the channel.
- For existing conditions sand and finer material (Silt, clay) erodes from the dredged material placement area and accumulates in the channel and inside the harbor. Small accumulation of sand occurs in the very vicinity of the jetty from the east side.

- Erosion for existing conditions occurs in deep water seaward of the harbor entrance. This result appears to be consistent with the general bathymetry of the area. The bottom depression located seaward of the jetty was observed in all available hydrographic surveys. Though this depression is slightly offset from the entrance in the Eastward direction, the general trend of scour shown in the model (depth of scour and orientation of the scour) indicates a similarity between the modeling results and typically observed field processes.
- Implementation most of the jetty extension alternatives (excluding Alternatives 2 and 3B) will not result in increase existing conditions of sedimentation and maintenance dredging requirements. Furthermore the alternatives may result in slight reduction of sedimentation in the channel and harbor due to reduction of westward transport of fine sediment.
- Alternative 2 may result in increase of sedimentation in the channel due to high gradient of bottom velocities due to the clearance under the structures. Sedimentation may include deposition of fine sediment as well as coarse sand and gravel.
- Alternative 3B may result in slightly increase of sedimentation due to the overflow of suspended sediment over the submerged portion of the jetty. Because of sedimentation would consists of suspended sediment that is sand and finer material, the added volume of sedimentation would be insignificant.
- Implementation of most alternatives (excluding Alternatives 3B and 4) will not result in loss of sand from the beach to the east of the jetty. Alternatives 3B and 4 may result in slight removal of sand relatively to existing conditions due to increase of the tidal velocities at local spots of the beach. However, this will not affect gravel and coarse sand sediment.
- All alternatives will result in the local scour at the toe of the jetty extension. This fact should be taken into consideration for design and cost estimate purposes. No preference with regard to a scour hole formation at the toe of the jetty was found for any alternative.

4.4. Water Quality Modeling

The objective of water quality modeling was to evaluate the effect of the proposed jetty construction alternatives on water quality in the Keystone

Harbor. It should be noted that certain water quality effects in the harbor, specifically in the Northern corner of the harbor, may be influenced by Crockett Lake since it is connected to the harbor by two large-diameter culverts. However, due to the lack of information required for numerical modeling of areas outside Keystone Harbor, including bathymetry and topography of Crockett Lake, lake water elevations, culvert dimensions and discharge characteristics, the Crockett Lake basin was not included into the modeling grid. Considering that water quality modeling was to provide a relative comparison between the alternatives and existing conditions, it is suggested that the modeling efforts performed herein are sufficient.

4.4.1. Model Description

Water quality modeling was conducted using the two-dimensional, finite element hydrodynamic model RMA4 (US Army Corps of Engineers Coastal and Hydraulics Laboratory 2001). The RMA4 model simulates depth-average dispersion, decay, and transport of conservative and non-conservative constituents under currents.

Alternatives were evaluated for performance in water quality by analysis of their estimated residence time. Residence time is a general measure of the rate of turnover in a water body, with lower residence time generally indicating better water quality. Residence time was evaluated using numerical dye flushing simulations using the results of tidal current circulation modeling as forcing. The results of the simulations are limited to qualitative and comparison analysis and can not be used for other purposes prior validation the model with field dye studies.

4.4.2. Model Domain Setup and Boundary Conditions

The RMA4 model domain covered the same area as the LAGRSED model discussed in Section 4.3. Figure 4.49 shows the RMA4 model domain and numerical grid overlaid with color depth contours for existing conditions as an example. Alternative grids were constructed for each alternative. A dye concentration measurement station was installed in the model in the rear of Keystone Harbor at the location of the culvert to extract depth-averaged concentrations during the simulation.

As it was discussed above the RMA4 model is forced with tidal currents that were simulated by the region-wide ADCIRC model. ADCIRC model results (water levels and velocities) from the large-scale finite element domain were interpolated onto the different small-scale RMA4 domain using custom finite element mapping routines. The period of simulation using RMA4 represents typical tidal flow conditions for Keystone Harbor area and corresponds to the range of tidal fluctuation observed from January 28 to February 09, 2004.

The RMA4 modeling approach was developed to determine changes on concentration in the water columns relatively to existing conditions. No estimates of absolute value of water concentrations were anticipated in the study. Therefore the initial concentration in simulation was described as 100% of existing value. Boundary conditions for the simulations were constant zero-concentration conditions along the ocean boundary. Dispersion coefficients were calculated automatically by the model based on grid and flow information, and were spatially variable. Simulations covered a seven-day period at 15-minute time steps.

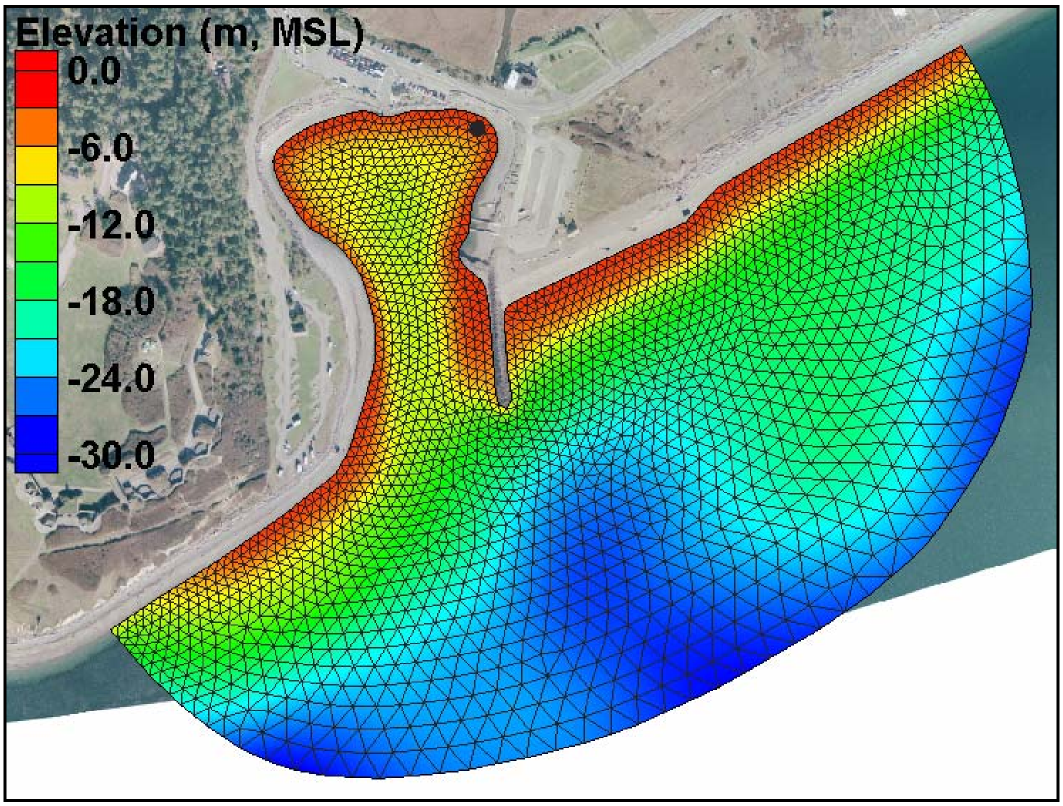

Figure 4.49 RMA4 Model Existing Conditions Finite Element Domain and Location of Concentration Measurements in Keystone Harbor

4.4.3. Model Results

The RMA4 model simulation included concentrations at the measurement point in the rear of Keystone Harbor as a function of time. Figure 4.50 shows concentrations in the domain at four different instant times during the simulation (t = 12, 24, 36 and 48 hours) for existing conditions. In general, the dye (indicator or concentration) is flushed out of the harbor within approximately 5 days. To calculate residence time, an exponential curve was fit to the time series of concentration in the rear of the harbor. Residence time was calculated as the reciprocal of the coefficient in the exponential equation. Figure 4.51 shows the concentrations as a function of time at the rear of the harbor for existing conditions and all jetty alternatives.

In order to compare the alternatives to the water quality criteria, a percent of change in residence time was calculated for all alternatives relatively to existing conditions. Table 4.12 below shows the relative residence time for each alternative, or the ratio between the residence time for the alternative and that for existing conditions.

Table 4.12 Residence Times for Jetty Alternatives Relative to Existing Conditions

Alternative	Change in Residence Time (%)
1	30%
1A	29%
2	17%
2A	31%
3	30%
3A	27%
3B	-9%
4	19%
4A	25%
5	-40%

Water quality modeling results indicate that the construction of the jetty extension and relocation alternatives alter the residence time relative to existing conditions, for some alternatives the volume of water turnover rate within Keystone Harbor may increase, while for other-reduces. It should be noted that residence time in reality is likely lower than in the model due to the lack of other meteorological and hydrodynamic factors, such as wind, waves, etc which typically tend to enhance water exchange.

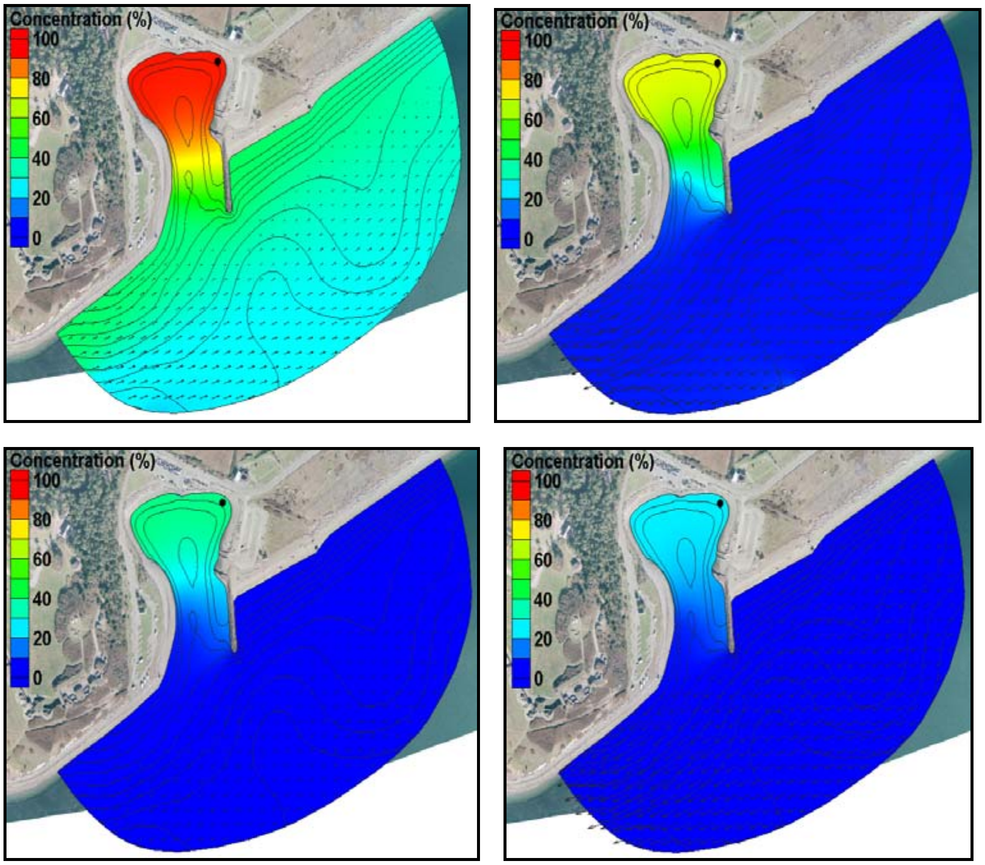

Figure 4.50 Existing Conditions Dye Concentrations at Four Different Simulation Times (upper left to bottom right: t = 12.0, 24.0, 36.0 and 48.0 hours)

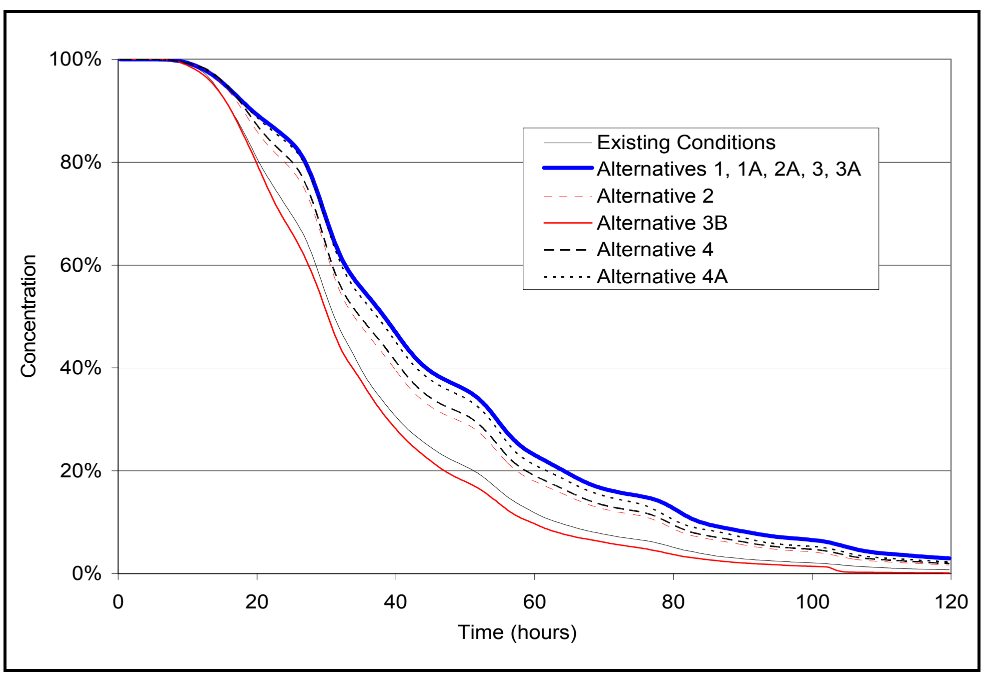

Figure 4.51 Concentrations as a Function of Time at Measurement Location for Existing Conditions and Jetty Alternatives

- Evaluation of the alternative with regard to the water quality criteria was based on computation residence time for each of the alternative.
- Results of computation show that most of jetty extension, including Alternative 3 may increase slightly the residence time. It implies on slight deterioration of the water quality. Observation on the model shows that quality of the water in the harbor (residence time) is inversely proportional to blockage by the jetty the upper water column area.
- Alternative 3B would reduce slightly the residence time in the harbor. This effect is due to the fact that the jetty extension is submerged. Lowering the jetty crest elevation would reduce residence time and improve the water quality in the harbor if requires. reduction of water
- Alternative 5, Jetty Relocation would reduce the residence time and most likely improve the water quality in the harbor. .

5. Keystone Physical Modeling

5.1. Objectives of Physical Modeling

Physical modeling of the Keystone Ferry Terminal and the jetty extension alternatives was conducted to meet the following goals:

- Verify and validate the numerical modeling of tidal flow circulation and wave transformation
- Qualitatively evaluate the alternatives with regard to their effect on shoreline changes in the vicinity of the jetty
- Quantitatively assess the effects of the jetty extensions on cross-channel currents at the entrance to Keystone Harbor

5.2. Modeling Facilities

The physical modeling was conducted in the three-dimensional wave basin at the O.H. Hinsdale Wave Research Laboratory (WRL) at Oregon State University. The basin is one of the largest facilities in the United States that equipped with most modern technologies and equipment to provide modeling of coastal processes in a high resolution scales. Tank dimensions are as follows. Length - 160 ft, width - 87 ft, depth - 7 ft. The picture of wave tank facilities during construction of the model is shown in Figure 5.1.